

logged 8-9-72

W R Hudson

RECEIVED AUG 09 1972

HIGHWAY RESEARCH RECORD

Number 390 | Geometric Design
Implications and
Vehicle Noise



HIGHWAY RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

1972

HIGHWAY RESEARCH BOARD

OFFICERS

Alan M. Voorhees, *Chairman*

William L. Garrison, *First Vice Chairman*

Jay W. Brown, *Second Vice Chairman*

W. N. Carey, Jr., *Executive Director*

EXECUTIVE COMMITTEE

A. E. Johnson, *Executive Director, American Association of State Highway Officials* (ex officio)

F. C. Turner, *Federal Highway Administrator, U.S. Department of Transportation* (ex officio)

Carlos C. Villarreal, *Urban Mass Transportation Administrator, U.S. Department of Transportation* (ex officio)

Ernst Weber, *Chairman, Division of Engineering, National Research Council* (ex officio)

D. Grant Mickle, *President, Highway Users Federation for Safety and Mobility* (ex officio, Past Chairman 1970)

Charles E. Shumate, *Executive Director, Colorado Department of Highways* (ex officio, Past Chairman 1971)

Hendrik W. Bode, *Gordon McKay Professor of Systems Engineering, Harvard University*

Jay W. Brown, *Director of Road Operations, Florida Department of Transportation*

W. J. Burmeister, *Executive Director, Wisconsin Asphalt Pavement Association*

Howard A. Coleman, *Consultant, Missouri Portland Cement Company*

Douglas B. Fugate, *Commissioner, Virginia Department of Highways*

William L. Garrison, *Edward R. Weidlein Professor of Environmental Engineering, University of Pittsburgh*

Roger H. Gilman, *Director of Planning and Development, Port of New York Authority*

George E. Holbrook, *E. I. du Pont de Nemours and Company*

George Krambles, *Superintendent of Research and Planning, Chicago Transit Authority*

A. Scheffer Lang, *Department of Civil Engineering, Massachusetts Institute of Technology*

John A. Legarra, *Deputy State Highway Engineer, California Division of Highways*

William A. McConnell, *Director, Product Test Operations Office, Product Development Group, Ford Motor Company*

John J. McKetta, *Department of Chemical Engineering, University of Texas*

John T. Middleton, *Deputy Assistant Administrator, Office of Air Programs, Environmental Protection Agency*

Elliott W. Montroll, *Albert Einstein Professor of Physics, University of Rochester*

R. L. Peyton, *Assistant State Highway Director, State Highway Commission of Kansas*

Milton Pikarsky, *Commissioner of Public Works, Chicago*

David H. Stevens, *Chairman, Maine State Highway Commission*

Alan M. Voorhees, *President, Alan M. Voorhees and Associates, Inc.*

Robert N. Young, *Executive Director, Regional Planning Council, Baltimore*

HIGHWAY RESEARCH RECORD

Number Geometric Design
390 Implications and
 Vehicle Noise

6 reports prepared for the
51st Annual Meeting

Subject Areas

- | | |
|----|--------------------------------|
| 15 | Transportation Economics |
| 22 | Highway Design |
| 26 | Pavement Performance |
| 52 | Road User Characteristics |
| 53 | Traffic Control and Operations |

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

Washington, D.C.

1972

NOTICE

The studies reported herein were not undertaken under the aegis of the National Academy of Sciences or the National Research Council. The papers report research work of the authors done at the institution named by the authors. The papers were offered to the Highway Research Board of the National Research Council for publication and are published herein in the interest of the dissemination of information from research, one of the major functions of the HRB.

Before publication, each paper was reviewed by members of the HRB committee named as its sponsor and was accepted as objective, useful, and suitable for publication by NRC. The members of the committee were selected for their individual scholarly competence and judgment, with due consideration for the balance and breadth of disciplines. Responsibility for the publication of these reports rests with the sponsoring committee; however, the opinions and conclusions expressed in the reports are those of the individual authors and not necessarily those of the sponsoring committee, the HRB, or the NRC.

Although these reports are not submitted for approval to the Academy membership or to the Council of the Academy, each report is reviewed and processed according to procedures established and monitored by the Academy's Report Review Committee.

ISBN 0-309-02060-3

Price: \$2.00

Available from

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

CONTENTS

FOREWORD	iv
INDEPENDENT VERSUS NARROW-MEDIAN ALIGNMENT: COMPARATIVE ECONOMY, SAFETY, AND AESTHETICS James S. Peet and Dennis Neuzil	1
HIGHWAY CURVE DESIGN FOR SAFE VEHICLE OPERATIONS John C. Glennon and Graeme D. Weaver	15
COST-EFFECTIVENESS TECHNIQUE FOR ANALYSIS OF ALTERNATIVE INTERCHANGE DESIGN CONFIGURATIONS Joseph A. Wattleworth and Jerry W. Ingram	27
DESIGN AND STRIPING FOR SAFE PASSING OPERATIONS Graeme D. Weaver and John C. Glennon	36
INSTABILITY ANALYSIS OF A VEHICLE NEGOTIATING A CURVE WITH DOWNGRADE SUPERELEVATION William Zuk	40
AUTOMOTIVE NOISE: ENVIRONMENTAL IMPACT AND CONTROL B. Andrew Kugler and Grant S. Anderson	45
SPONSORSHIP OF THIS RECORD	56

FOREWORD

All highway engineers engaged in design, public safety, and environmental control should find the articles in this RECORD interesting and provocative. The paper by Glennon and Weaver should be read in sequence with the one by Zuk; these 2 unrelated and independent studies on highway curve design reach parallel conclusions. The controversial subject of the relative effectiveness of the 3 types of automobile restraint systems should be read by the general public as well as by engineers.

Peet and Neuzil discuss the relative merits of independent alignments versus constant median alignment for a high type of roadway design. The authors include economic analysis and graphs to prove that, within certain limits, independent alignment can often be built and maintained at less total annual cost than some narrow-median designs.

The Texas Transportation Institute conducted a series of field studies to test whether vehicles follow the path of a highway curve with geometric exactness. Glennon and Weaver discuss the procedures followed and the mathematical analysis of the findings. Results show that the path of a certain percentage of vehicles will exceed that of the highway curve. The authors thereby proceeded to develop a hypothetical highway curve design equation to provide greater safety margin against lateral skidding.

Wattleworth and Ingram present a case study to illustrate a cost-effectiveness methodology for analyzing and comparing alternate interchange configurations and for determining a phasing program of future improvement.

Weaver and Glennon challenge the adequacy of current passing sight distance criteria and striping practices, which were based on studies conducted between 1938 and 1941. In their abridged paper, they describe field studies that were conducted to examine passing behavior on rural 2-lane highways and to develop passing sight distance design criteria compatible with current operating conditions. Of primary concern were passing maneuvers on highways with operating speeds of 50 to 80 mph. A new concept is presented that integrates design and striping.

Zuk presents a mathematical analysis of a vehicle negotiating a curve with downgrade superelevation. The study was initiated as a result of numerous skidding accidents at locations where highway geometrics include a combination of downgrade, curve, and superelevation. The author concludes that lateral coefficient of friction and driver maneuvering are the most important factors in critical skidding.

Kugler and Anderson discuss units used to describe automotive noise, current criteria used in assessing the impact of traffic noise on people, various automotive sources that combine to create traffic noise, and control of automotive noise. An example of noise control through highway design is presented.

—B. H. Rottinghaus

INDEPENDENT VERSUS NARROW-MEDIAN ALIGNMENT: COMPARATIVE ECONOMY, SAFETY, AND AESTHETICS

James S. Peet, Hershey, Malone and Associates; and
Dennis Neuzil, Tippetts-Abbett-McCarthy-Stratton

Independent alignments and other wide-median designs are generally superior to narrow-median designs from the standpoint of safety and aesthetics. For a wide range of design and cost conditions often associated with rural and suburban highway locations, the liberal-median designs are economically competitive with narrow-median designs and deserve increased consideration by the highway design team. Annual costs of selected independent alignment and narrow-median designs are presented for a range of cost and terrain factors. Safety performance requirements are incorporated in a comparison chart that permits rapid, preliminary economic evaluation of typical alternative median designs.

- TOO OFTEN minimum geometric design standards and design guides appear to have been adopted as basic working standards from which departures toward more liberal treatments are assumed to require excessive additional costs. The significance of relations among design features, highway aesthetics, and motoring safety is not always fully appreciated. Common examples are the use of steep side slopes with roadside guardrail instead of unprotected flat slopes (1), short vertical and horizontal curvature instead of longer and more flowing curvature, and narrow medians with or without median barriers instead of wide, variable width of independent alignment. It is increasingly recognized, however, that the development of more attractive and safer highways requires more than minimum design treatments and that aesthetic and safety factors must be given weightings comparable to economy in the design decision process.

The following sections contain a review of the relative advantages of independent and narrow-median alignment designs and of the criteria necessary for comprehensive evaluation of these alternative designs with regard to economy, safety, aesthetics, and environmental impact. Annual costs of selected independent and narrow-median designs are presented for a range of cost and terrain factors. Economic comparisons are facilitated by charts showing "economic break-even widths" for designs with wide medians—the additional median width that can be added to the minimum design treatment for independent alignment without exceeding the annual cost of an alternative narrow-median design.

COMPARATIVE ADVANTAGES AND DESIGN CRITERIA

The independent alignment consists of roadways for which horizontal and vertical alignments are developed individually to suit location and design requirements; the narrow-median alignment employs the same horizontal alignment and profile for both roadways (although in some cases the latter treatment has been used with rather wide medians in flat terrain locations). On hillside locations the independent alignment places each roadway at a different elevation, and the narrow-median alignment places each roadway at or near the same elevation at a constant distance apart (Fig. 1).

Economics

The most economical design is the one that has the lowest total annual cost (not necessarily the lowest initial cost); therefore, annual maintenance and road-user costs should be included in the economic analysis. In addition to the major initial costs of earthwork and right-of-way, costs of items such as topsoil, seeding, median and roadside barrier installation, mowing, and maintenance of slopes, culverts, and ditches should also be included in the analysis.

Independent alignment will usually require less earthwork but more right-of-way than the narrow-median design. With its wide, varying median width and narrower grading prisms, independent alignment often permits the designer to avoid areas of difficult topographic, soil, or drainage conditions. The shallower cuts of the independently aligned roadways can mean less likelihood of slope failures and erosion, groundwater problems, and bedrock excavation. The construction of 2 independent roadways widely separated may, however, require more nonproductive equipment movements than would occur in the construction of a narrow-median alignment.

The relative advantages of the 2 types of alignment designs with respect to structure and drainage facility costs will depend on the specific terrain and drainage conditions. Narrowing a wide median where bridge and grade separation costs would otherwise be excessive is not inconsistent with the variable-median-width concept of independent alignment. Independent alignments may increase the number and total length of culverts required, although the wide, varying median may permit taking better advantage of easier stream-crossing sites.

The narrow-median design may require a median barrier to provide adequate safeguard against cross-median collisions, and the independent alignment may similarly require a median guardrail on the upper roadway. The wide median provides an ample reserve for the addition of more traffic lanes should estimates of future traffic be low; the cost of such expansion is negligible compared to the construction of a new, wider highway. When highways with narrow-to-intermediate medians are provided with additional lanes in the median, an additional expenditure must usually be made for a median barrier.

The independent alignment will often yield a number of road-user cost savings not yielded by the narrow-median design. The maintenance and repair of median barriers on highways with narrow medians often require closing median lanes, and the resulting increased user cost is properly assignable to the narrow-median design. According to AASHO, there is another potential benefit to road users from independent alignment (2): "By varying the separation between the two roadways, it is often possible to improve the profile over crests by using a flatter grade on the ascent than on the descent for each direction of travel." Increased traffic capacity is also obtained by this treatment. With independent or wide-median designs, a roadway can be opened to traffic sooner and thus provide earlier service for traffic demands and the attendant user cost savings. If a second roadway is to be provided adjacent to an existing highway to form a divided highway, a wide-median design will result in less interference to traffic during construction and provide both cost and safety benefits. For divided highways with at-grade intersections, the wide median can sometimes obviate the need for traffic signals at low-volume crossroads by enabling 2-step crossing maneuvers. U-turns are also accommodated with less disruption to through traffic.

Accident costs should desirably be included in the economic analysis of alternative designs, although they can be incorporated indirectly in the evaluation procedure as noted below. Aesthetic and environmental costs—less tangible but no less real than the traditional highway economic costs—should also be included in the design decision-making process.

Safety and Driver Comfort

Wide medians improve personal comfort and general motoring safety by reducing the annoyances of headlight glare, buffeting, and noise from traffic on the opposing roadway. But most important, they lessen the chances for cross-median collisions where median barriers are absent.

Figure 1. Basic cross-sectional designs.

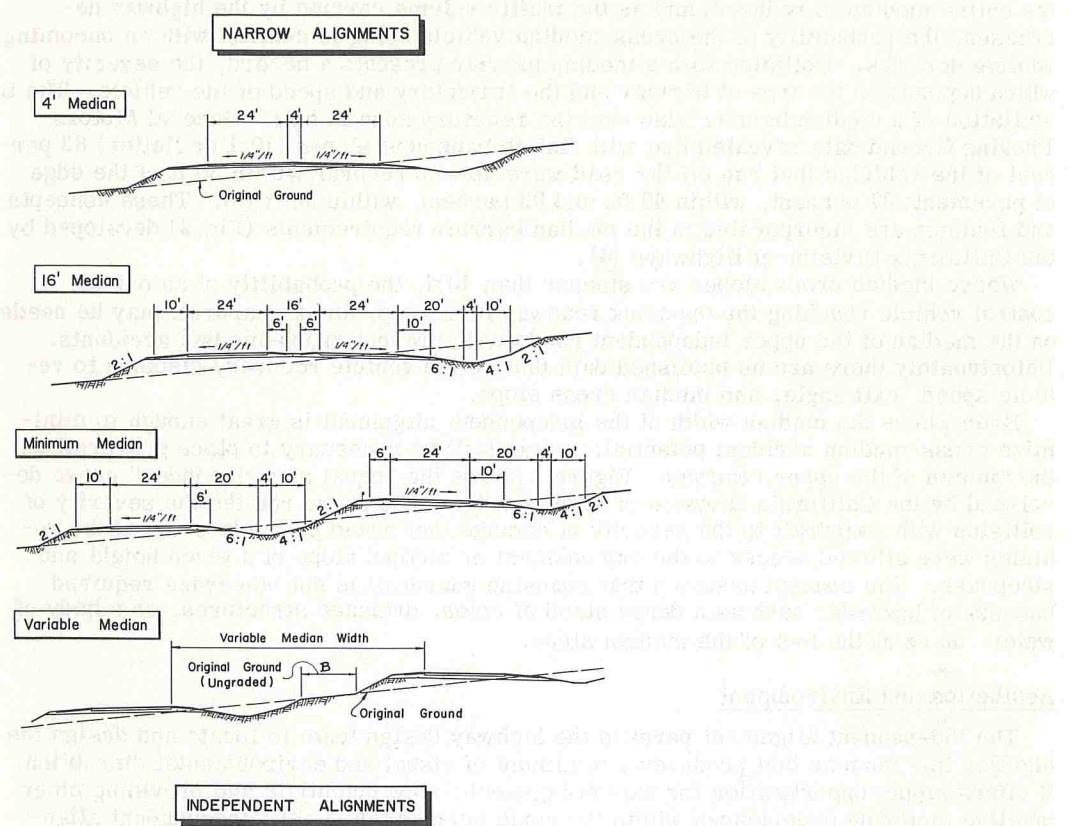


Figure 2. Median barrier requirements.

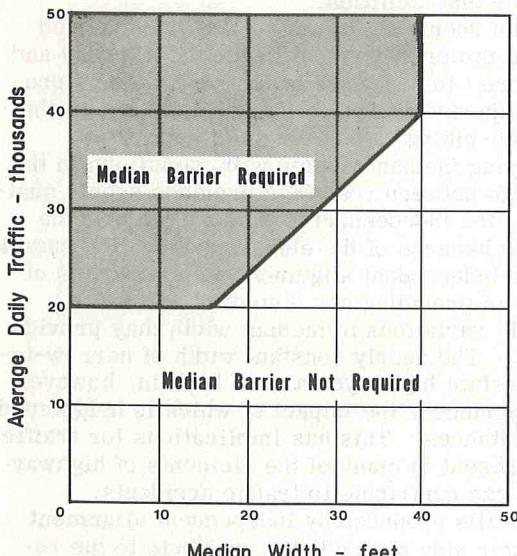
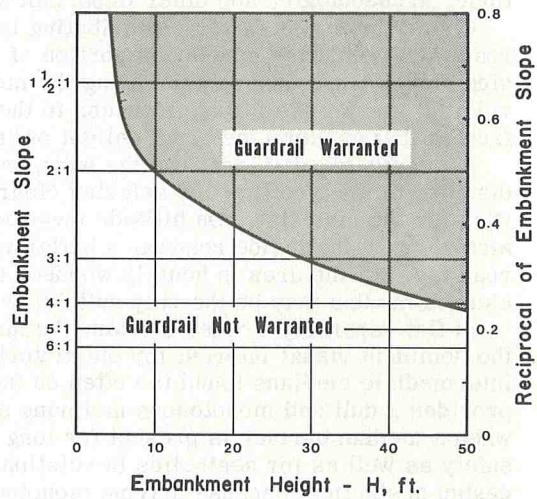


Figure 3. Equal severity curve.



As median width is increased the probability of an out-of-control vehicle crossing the entire median is reduced; and as the traffic volume carried by the highway decreases, the probability of the cross-median vehicle being in conflict with an oncoming vehicle declines. Collision with a median barrier presents a hazard, the severity of which depends on the type of barrier and the trajectory and speed of the vehicle. The installation of a median barrier also cuts the recovery zone in half. General Motors Proving Ground data revealed that with flat embankment slopes (10:1 or flatter) 83 percent of the vehicles that ran off the road were able to recover within 30 ft of the edge of pavement; 87 percent, within 40 ft; and 93 percent, within 50 ft (3). These concepts and findings are incorporated in the median barrier requirements (Fig. 2) developed by the California Division of Highways (4).

Where median cross slopes are steeper than 10:1, the probability of an out-of-control vehicle reaching the opposing roadway increases, and a guardrail may be needed on the median of the upper independent roadway to prevent cross-median accidents. Unfortunately there are no published data that relate vehicle recovery distance to vehicle speed, exit angle, and median cross slope.

Even where the median width of the independent alignment is great enough to minimize cross-median accident potential, it may still be necessary to place guardrail on the median of the upper roadway. Figure 3 shows the "equal severity index" curve developed by the California Division of Highways (5). The curve relates the severity of collision with guardrail to the severity of damage that would probably occur if the vehicles were allowed access to the embankment or median slope of a given height and steepness. The concept assumes that roadside guardrail is not otherwise required because of hazards, such as a dense stand of trees, drainage structures, or a body of water, on or at the foot of the median slope.

Aesthetics and Environment

The independent alignment permits the highway design team to locate and design the highway in a manner that produces a minimum of visual and environmental disruption. It offers ample opportunities for exploiting scenic-view potentials and providing other positive motoring experiences within the route corridor (6, 7, 8). Independent alignment provides greater opportunities for attractive landscaping and retaining native vegetation and imparts a parkway appearance even to the heavily trafficked, general purpose highway. In some cases its use can reduce or even eliminate disruption to historic, archaeologic, and other important sites and facilities.

One of the major factors contributing to the scenic character of widely separated roadways is that the man-made portion of the motorist's visual field—the roadway and side slopes—is reduced, permitting the motorist to see more of the roadside and providing a greater feeling of closeness to the adjacent landscape (6). This is one of the factors that explains the great delight of "shun-piking" along the quiet byway.

In heavily wooded locations the wide, varying median coupled with variations in the distance to the tree line and selected clearings between roadways provides scenic qualities for the motorist. On hillside locations, the independent alignment can provide motorists on the inside roadway a better view because of the elevation above the outside roadway. (If the area is heavily wooded, the independent alignment with stretches of cleared median may be the only suitable way of providing any distant views at all.)

In flat, sparsely wooded terrain, "natural" variations in median width may provide the dominant visual interest for the motorist. The nearly constant width of narrow-to-intermediate medians found too often on Interstate highways in such terrain, however, provides a dull and monotonous motoring experience, the impact of which is heightened when a median barrier is present for long distances. This has implications for traffic safety as well as for aesthetics (a relation present in many of the elements of highway design aesthetics) because driving monotony can contribute to traffic accidents.

On hillside locations the smaller cuts and fills produced by independent alignment and the attendant increased feasibility of flatter side slopes both contribute to the reduction of the visual impact of the terrain "scar." Unsightly erosion, with its adverse effects on surface water quality, is also reduced.

The man-made character of median barriers often used with narrow and intermediate medians is an aesthetic disruption. When they are kinked, damaged, or rusted, their unattractiveness is intensified. Vegetation is often ragged beneath rail barriers, accentuating the unattractiveness of the installation. (These characteristics apply equally to roadside guardrail.) More important, the median barrier creates a discontinuity in the smooth flow of the highway cross section, chopping up the roadway space and interfering with the attainment of a smooth merge of highway and surrounding landscape. It gives the motorist a feeling of confinement or lateral friction, especially when guardrail is present along the outside shoulder.

The wide median, particularly if wooded, probably reduces the frequency of vehicle-struck wildlife (especially deer) by providing an ample "recovery area" for crossing animals. Where development is likely to occur along a new rural or suburban highway, the use of a wide median will reduce the intensity of noise adjacent to the roadways because traffic density is reduced. The wide median also permits the planting of trees and shrubs that tend to reduce perceived noise levels.

CROSS-SECTIONAL MODELS

Several 4-lane cross sections were selected as reflecting a typical range of alternative designs. Figure 1 shows examples of typical narrow-median and independent-alignment designs studied. Cut-and-fill slopes are shown as 2:1, but designs with 4:1 slopes were also treated. Median width is the distance between inside pavement edges. The narrow-median design is shown for the 4-ft and 16-ft width cases. For the general case of independent alignment, the roadway earthwork prisms are separated by natural ground of varying width B . For the minimum median width of independent alignment, the 2 earthwork prisms abut ($B = 0$). Shoulders are 10 ft on the outside and 6 ft on the inside (median side), except that median shoulders are omitted in the case of the 4-ft narrow-median design. Where roadside guardrail is required, the width of the shoulder is increased by 4 ft to provide barrier post support and an effective shoulder width equivalent, after rounding, to that provided by side slopes 4:1 or flatter without barriers (2, 9).

On hillside locations the roadways of the independent alignment were placed so that earthwork was in balance for each individual roadway. For the narrow-median case, the earthwork balance was based on the entire roadway cross section (traveled ways plus shoulders plus median). Cut volumes, less 10 percent allowance for shrinkage, were balanced against fill volumes, and haul charges were neglected. This method appears to be quite reasonable when cut-and-fill slopes are equal and when the unit cost of excavation is approximately equal to the unit cost of borrow.

In this study it was assumed that the unit cost of excavation included the cost of placing excavated material as fill. Where substantial amounts of rock are present, it might be cheaper to minimize excavation and to import large volumes of borrow, placing most of the roadway on embankment.

Figure 4 shows the minimum median widths attained by independent alignments on hillside locations for the given cross sections and their placement on hillsides according to the method of earthwork balancing described above. Minimum median widths are quite substantial even for the case of 2:1 roadside slopes, exceeding 50 ft for original ground cross slopes of about 5 percent or greater.

EARTHWORK AND RIGHT-OF-WAY QUANTITIES

Differences in earthwork and right-of-way quantities largely determine the relative economy of independent-alignment and narrow-median designs without median barrier. Figure 5 shows a plot of excavation volumes for a 1-ft slice of a 16-ft narrow-median design and the independent-alignment design as a function of original ground cross slope (hereafter referred to as "ground slope"). Selected minimum median widths produced by the independent alignments (Fig. 4) have been indicated on their curves. The difference in excavation volumes between the narrow and independent alignments increases with increasing ground slope and is greater for designs with the flatter 4:1 side slopes. For both 2:1 and 4:1 slopes, the narrow-median excavation exceeds that of the indepen-

dent alignment by 25 to 30 percent for ground slopes of 10 percent or greater. An analysis of the effect of median width on excavation differences showed that 4-ft and 40-ft narrow-median designs required respectively about 3 percent and 90 percent more excavation than independent alignment for a ground slope of 10 percent.⁷ Figure 5 also shows that at flatter ground slopes (less than 10 percent) there is very little difference in excavation quantities between a design with a 2:1 side slope and one with the more aesthetic and maintenance-free 4:1 side slope.

Figure 6 shows right-of-way requirements for the 16-ft narrow-median and independent-alignment designs for 2:1 and 4:1 side slopes. The border component of the right-of-way (adjacent to the outer cut-and-fill slopes) has been omitted but would be the same for both narrow-median and independent-alignment designs. The right-of-way of the independent alignment exceeds that of the narrow median for all conditions, with the percentage difference in right-of-way increasing to about 25 percent at steeper ground slopes.

ECONOMIC ANALYSES: DESIGNS WITHOUT BARRIER RAILS

Annual Costs

Comparisons were made of the total annual cost of alternative narrow-median and independent-alignment designs based on typical construction costs and maintenance costs, the latter generally representative of conditions and practices in northern, humid areas (1). A 20-year economic life for initial cost items and a 6 percent interest rate were used in all analyses.

Figure 7 shows total annual costs of several alternative median designs as a function of ground slope. An excavation cost of \$1/yd³ and a right-of-way cost of \$1,000/acre were used. Other construction and maintenance costs are shown in the figure. Median barriers and roadside guardrails were not included with these designs. For these unit costs, independent alignment is economically competitive with the 16-ft narrow-median design and is quite superior from an economic standpoint to the 40-ft narrow-median design. For example, at a ground slope of about 12 percent, the annual costs of the 16-ft narrow-median and independent-alignment designs are equal (4:1 side slope case), yet the latter provides a median 100 ft wide. At the same ground slope, independent alignment costs about 30 percent less than the 40-ft median but provides a median 2½ times greater than the latter.

Table 1 gives the effect of excavation cost on the relative economy of independent alignment and 16-ft narrow-median design for the 4:1 side slope case. Other costs are the same as those given in Figure 7. As expected, both increased unit cost of excavation and steeper ground slopes tend to favor independent alignment.

Break-Even Analysis

For the examples shown in Figure 7, independent alignment is more expensive than several alternative narrow-median designs at flat ground slopes but is cheaper at steep ground slopes. The ground slope at which the annual costs of alternative independent and narrow-median designs are equal can be referred to as the "break-even" ground slope. At ground slopes steeper than the break-even ground slope, the savings afforded by the use of independent alignment could be used to acquire additional right-of-way for the purpose of widening the median of the independent-alignment design. If the full amount of the savings is so used (bringing the annual cost of the independent alignment up to that of the narrow-median alternative), the total additional median width provided can be called the economic break-even median width. Total median width thus provided by the independent-alignment alternative would consist of the graded (minimum) median width (Fig. 4) plus the ungraded break-even median width. Break-even median widths will, of course, depend on unit costs and the geometry of the alternative designs of the cross section.

If various unit costs of right-of-way are applied to annual cost differences as developed previously, a break-even chart such as Figure 8 can be developed in which break-even median width is related to ground slope and unit right-of-way cost. (A family of such charts could be prepared for other cost variables, such as excavation cost.) As

Figure 4. Minimum median widths for balanced-earthwork independent alignments.

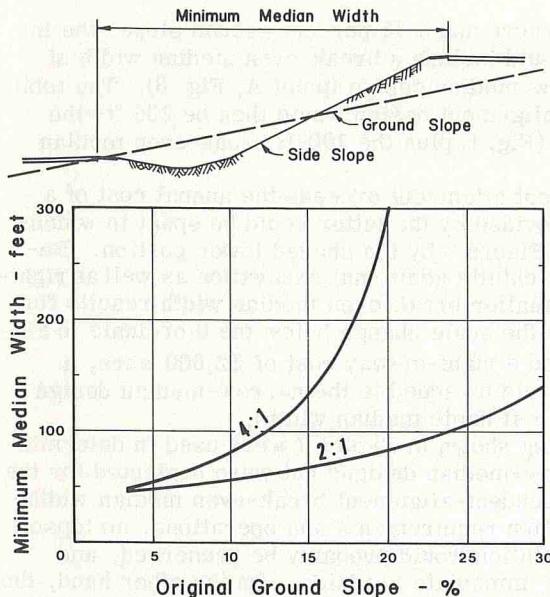


Figure 6. Minimum right-of-way required for selected median designs.

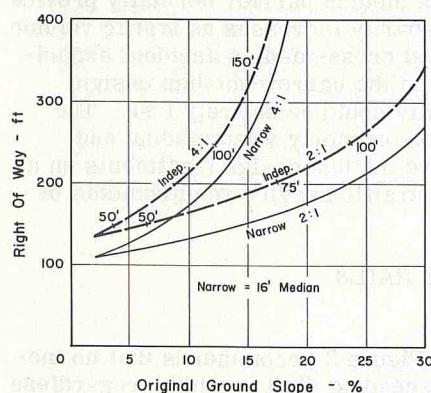


Table 1. Annual costs of independent alignment and 16-ft narrow-median designs at various unit excavation costs (4:1 side slopes).

Ground Slope (percent)	Annual Cost (\$) at \$1/Yd ³			Annual Cost (\$) at \$2/Yd ³		
	Independent	Narrow Median	Difference	Independent	Narrow Median	Difference
5	0.96	0.83	0.13	1.21	1.15	0.06
10	1.67	1.59	0.08	2.28	2.40	-0.12
15	3.10	3.14	-0.04	4.43	4.91	-0.48
20	7.39	7.77	-0.38	10.90	12.45	-1.55

Figure 5. Excavation volumes for selected median designs.

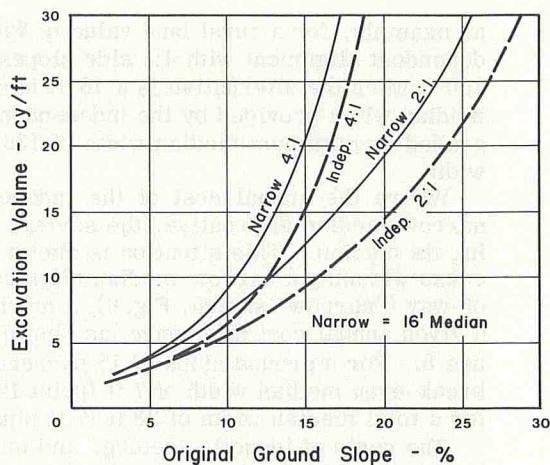
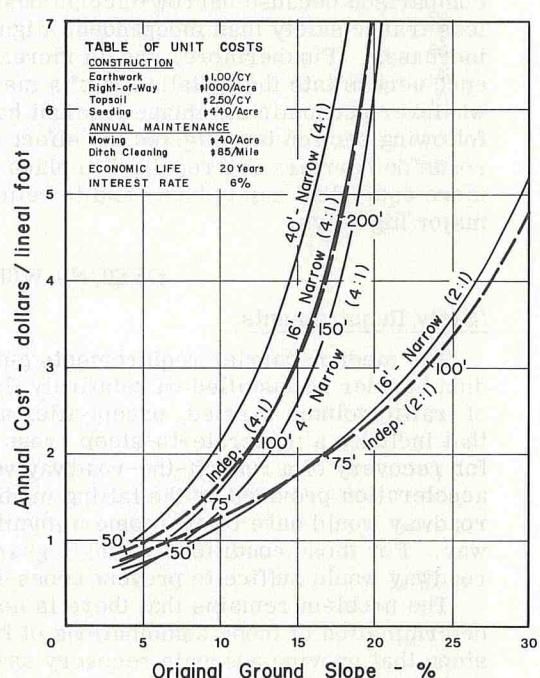


Figure 7. Annual costs of selected median designs.



an example, for a rural land value of \$500/acre and a 15 percent ground slope, the independent alignment with 4:1 side slopes could include a break-even median width of 100 ft when the alternative is a 16-ft narrow-median design (point A, Fig. 8). The total median width provided by the independent-alignment design would then be 236 ft—the graded or minimum median width of 136 ft (Fig. 4) plus the 100-ft break-even median width.

Where the annual cost of the independent alignment exceeds the annual cost of a narrow-median alternative, the savings provided by the latter could be spent in widening its median. This situation is shown in Figure 8 by the shaded lower portion. Because widening a narrow-median alignment entails additional excavation as well as right-of-way ("narrow" sketch, Fig. 8), a much smaller break-even median width results for a given annual cost difference, as shown by the scale change below the 0 ordinate in Figure 8. For a ground slope of 15 percent and a right-of-way cost of \$2,000/acre, a break-even median width of 7 ft (point B) could be added to the narrow-median design for a total median width of 23 ft (7 ft plus 16-ft basic median width).

The costs of topsoil, seeding, and mowing shown in Figure 7 were used in determining break-even median widths for the narrow-median designs but were neglected for the independent alignments. Because the independent-alignment break-even median width can usually be left undisturbed by construction requirements and operations, no topsoil or seeding would be required; existing vegetation would probably be preserved, and mowing operations would be confined to the immediate roadside. On the other hand, the entire width of the narrow-median design is disturbed by construction, making topsoil and seeding operations imperative. Relatively narrow medians usually require mowing for appearance and for control of saplings that might become hazardous.

The foregoing analysis does not fully meet the requirements of a valid economic comparison because narrow-median designs lacking a median barrier normally provide less traffic safety than independent alignment; the disparity increases as traffic volume increases. Furthermore, should increased traffic and cross-median accident experience necessitate the installation of a median barrier on the narrow-median design, whatever economic advantage it might have had initially could be entirely lost. The following section investigates the effect on comparative economy when median and roadside barriers are required to place the alternative median design treatments on a more equivalent safety basis and to better reflect the traffic service requirements of major highways.

DESIGNS WITH BARRIER RAILS

Safety Requirements

The median-barrier requirements guide shown in Figure 2 recommends that no median barrier be installed on relatively flat medians exceeding 40 ft in width, regardless of traffic volume carried, except after adverse accident experience. A 40-ft median that includes a moderate-to-steep cross slope would probably not provide enough space for recovery of a run-off-the-roadway vehicle from the upper roadway because of the acceleration provided by the falling median slope. However, vehicles from the lower roadway would have to overcome a significant grade in order to reach the upper roadway. For those conditions, a single guardrail along the inside shoulder of the upper roadway would suffice to prevent cross-the-median collisions.

The problem remains that there is no firm empirical basis available for accurate determination of those combinations of traversable median width and steeper median slope that provide adequate recovery space for vehicles leaving the upper roadway at high speeds. The General Motors Proving Ground data cited earlier and the California median-barrier requirements chart provide only a partial basis for approaching this problem. Nevertheless, for purposes of simplicity it has been assumed for median slopes in the range of 2:1 to 4:1 that 100 ft of relative obstacle-free median width will provide ample recovery space in nearly all cases. (The GM data showed that 93 percent of the run-off-the-roadway vehicles recovered on flat side slopes within 50 ft of the edge of pavement, or half this distance.) Some degree of safety factor is present in this assumption because the 14 ft of rising foreslope of the lower roadway plus its ad-

Figure 8. Break-even median width as a function of right-of-way cost for independent alignment versus 16-ft narrow-median design (4:1 side slopes).

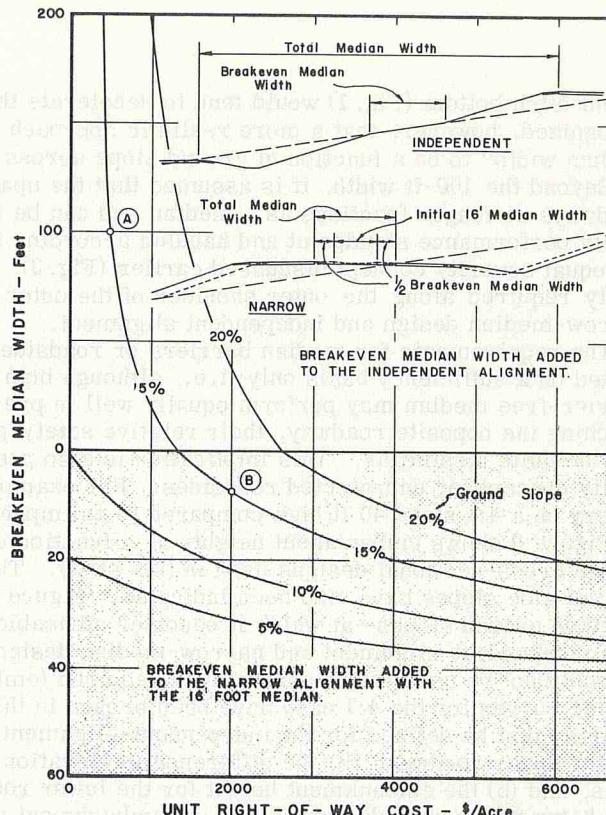
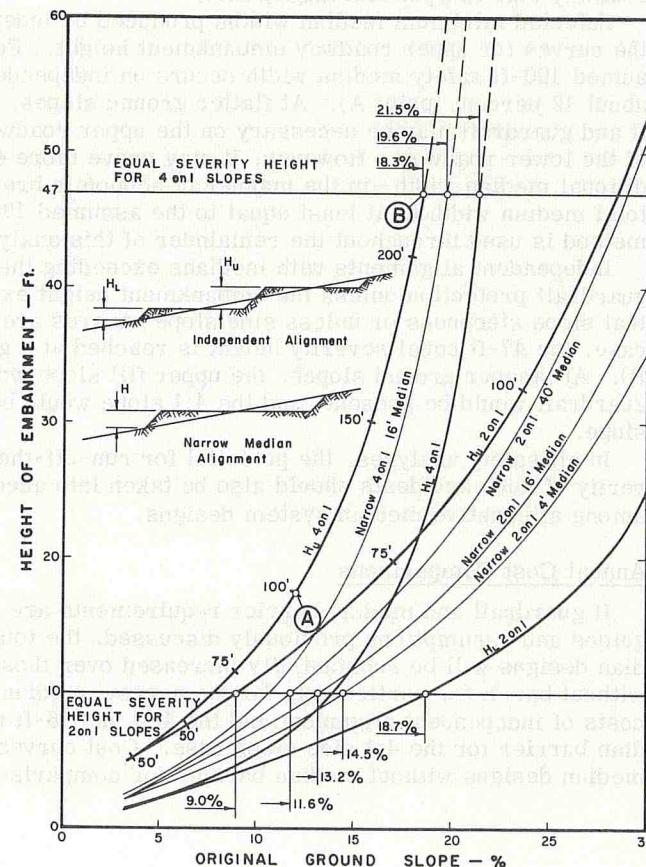


Figure 9. Roadway embankment heights.



jacent ditch bottom (Fig. 1) would tend to decelerate the vehicle somewhat. It is fully recognized, however, that a more realistic approach would show this "minimum safety median width" to be a function of ground slope across the median.

Beyond the 100-ft width, it is assumed that the space between the upper and lower roadways no longer functions as a median and can be treated as "roadside" from the safety performance standpoint and handled according to roadside hazard warrants and the equal severity concept discussed earlier (Fig. 3). Roadside guardrail may be similarly required along the outer shoulder of the outer (downhill) roadway on both the narrow-median design and independent alignment.

The requirements for median barriers or roadside guardrails are essentially presented on a sufficiency basis only; i.e., although both a median barrier and a wide barrier-free median may perform equally well in preventing most errant vehicles from reaching the opposite roadway, their relative safety performance for the errant vehicle may be quite dissimilar. This implication is also present in direct use of the equal severity concept for unprotected roadsides. For example, how much safer is vehicle recovery on a 4:1 slope 40 ft high compared to an impact with guardrail (Fig. 3)?

Figure 9 shows embankment heights as a function of ground slope for several of the 4-lane cross-sectional designs used in this study. The equal severity heights for 2:1 and 4:1 side slopes have also been indicated. Figure 9 shows the embankment heights—and thus ground slopes—at which it becomes advisable to install roadside guardrail on both independent-alignment and narrow-median designs. Because 4:1 slopes would probably not be used where guardrail is required (embankment height of more than 47 ft), the curves for the 4:1 case have been broken in this region. Two embankment heights must be defined for the independent-alignment case: (a) the embankment height for the upper roadway, H_u , the difference in elevation between the upper and lower roadways, and (b) the embankment height for the lower roadway, H_l , the difference in elevation between the shoulder break and the embankment toe. For a given side slope and ground slope, narrow-median designs have a higher embankment height on the outer roadway than independent alignments.

Selected minimum median widths produced by independent alignment are marked on the curves for upper roadway embankment height. For the 4:1 side slope case, the assumed 100-ft safety median width occurs on independent alignment at a ground slope of about 12 percent (point A). At flatter ground slopes, the median width falls below 100 ft and guardrail may be necessary on the upper roadway because of the hazard potential of the lower roadway. However, it may prove more economical to simply provide additional median width—in the manner of economic break-even median width—so that the total median width is at least equal to the assumed 100-ft safety median width. This method is used throughout the remainder of this analysis.

Independent alignments with medians exceeding the 100-ft safety width do not require guardrail protection unless the embankment height exceeds the equal severity height for that slope steepness or unless side slope hazards are present. In the 4:1 side slope case, the 47-ft equal severity height is reached at a ground slope of 18 percent (point B). At steeper ground slopes, the upper fill slope would probably be steepened because guardrail would be present, and the 4:1 slope would be approaching the existing ground slope.

In all safety analyses, the potential for run-off-the road accidents and the likely severity of such accidents should also be taken into account when decisions are made among alternative median system designs.

Annual Cost Comparisons

If guardrail and median barrier requirements are determined on the basis of the guides and assumptions previously discussed, the total annual costs of the various median designs will be substantially increased over those shown in Figure 7 for designs without barriers, particularly for the narrow-median designs. Figure 10 shows annual costs of independent alignment and the 4-ft and 16-ft narrow-median designs with median barrier for the 4:1 side slope case. Cost curves are also plotted for the narrow-median designs without median barrier for comparison. Right-of-way cost is taken at

Figure 10. Annual costs of selected median designs, with and without median barrier (4:1 side slopes).

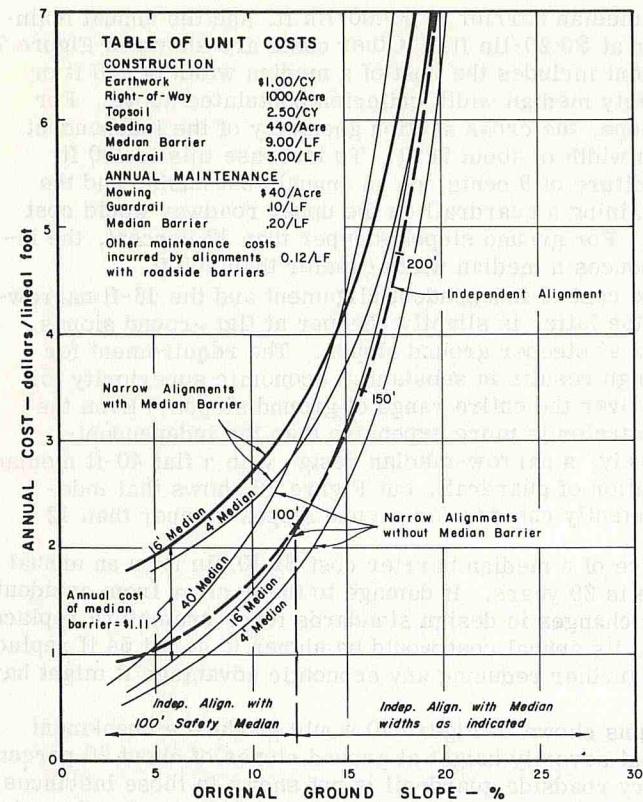


Figure 11. Annual costs of independent-alignment design and narrow-median designs with median barriers (guardrail included where warranted by equal severity curve, 2:1 side slopes).

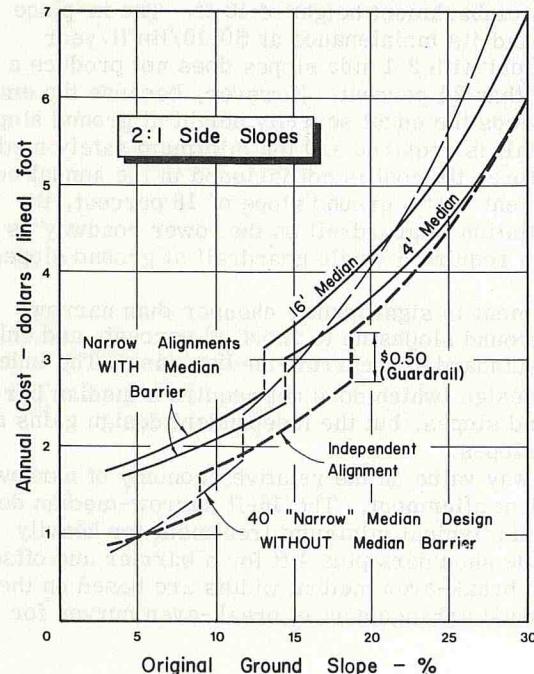
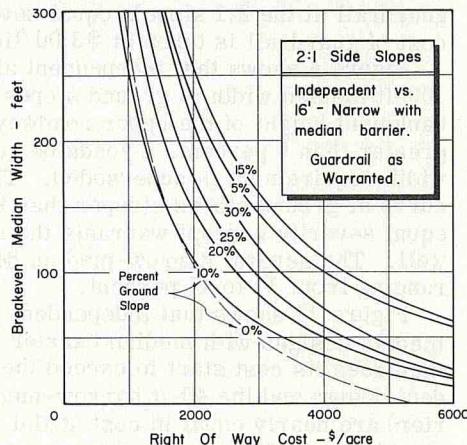


Figure 12. Break-even median width as a function of right-of-way cost for independent alignment versus 16-ft narrow-median design with median barrier (guardrail included where warranted by equal severity curve, 2:1 side slopes).



\$1,000/acre, the in-place cost of median barrier at \$9.00/lin ft, and the annual maintenance cost of the median barrier at \$0.20/lin ft. Other costs are shown in Figure 7.

The cost of independent alignment includes the cost of a median width of 100 ft or greater in accordance with the safety median width criterion postulated above. For example, at a 5 percent ground slope, the cross section geometry of the independent alignment would produce a median width of about 56 ft. To increase this to 100 ft would require an additional expenditure of 9 cents, on an annual cost basis, and the alternative of installing and maintaining a guardrail on the upper roadway would cost about 46 cents on an annual basis. For ground slopes steeper than 12 percent, the independent alignment naturally produces a median width greater than 100 ft.

There is little difference in the cost of independent alignment and the 16-ft narrow-median design without barriers—the latter is slightly cheaper at flat ground slopes, and the former is slightly cheaper at steeper ground slopes. The requirement for a median barrier on the 16-ft design results in substantial economic superiority for the independent-alignment design over the entire range of ground slopes. Even the narrow 4-ft median with median barrier is more expensive than the independent-alignment alternative. Alternatively, a narrow-median design with a flat 40-ft median could be used to avoid the installation of guardrail, but Figure 10 shows that independent alignment would be consistently cheaper for ground slopes steeper than 12 percent.

The installation and maintenance of a median barrier cost \$1.10/lin ft on an annual basis assuming that its useful life is 20 years. If damage to the barrier from accidents or poor maintenance practices or changes in design standards force premature replacement of large sections of barrier, its annual cost would be higher (e.g., \$1.54 if replaced at the same cost after 10 years), further reducing any economic advantage it might have had originally.

Several of the alternative designs shown in Figure 10 would produce embankment heights greater than the 47-ft equal severity height at ground slopes of about 20 percent. However, the cost of the necessary roadside guardrail is not shown in those instances because the extremely rapid rise in total annual cost at ground slopes greater than about 15 percent would no doubt require use of side slopes steeper than 4:1. The effect of roadside guardrail requirements on annual cost is more clearly shown in the case of 2:1 side slopes (Fig. 11).

Figure 11 shows annual costs of the alignment designs with the steeper 2:1 side slopes, again with the same unit costs. The vertical steps that occur in each curve represent the increases in annual cost resulting from the installation and maintenance of guardrail at the 2:1 slope's equal severity embankment height of 10 ft. The in-place cost of guardrail is taken at \$3.00/lin ft and its maintenance at \$0.10/lin ft/year.

Figure 4 shows that independent alignment with 2:1 side slopes does not produce a 100-ft median width at ground slopes less than 24 percent. However, because the embankment height of the upper roadway exceeds the equal severity height at ground slopes greater than 9 percent, a roadside guardrail is required and the minimum safety median width requirement is superseded. Therefore, its cost is not included in the annual cost curve at ground slopes steeper than 9 percent. At a ground slope of 18 percent, the equal severity concept warrants the installation of guardrail on the lower roadway as well. The several narrow-median designs require a single guardrail at ground slopes ranging from 11 to 15 percent.

Figure 11 shows that independent alignment is significantly cheaper than narrow-median designs with median barrier for ground slopes up to about 30 percent, and only then does its cost start to exceed the substandard 4-ft narrow-median case. The independent design and the 40-ft narrow-median design (which does not require a median barrier) are nearly equal in cost at flat ground slopes, but the independent design gains a sizable cost advantage at steeper ground slopes.

Figure 12 shows the effect of right-of-way value on the relative economy of narrow-median designs with barrier and independent alignment. The 16-ft narrow-median design is used because it could be considered a typical minimum treatment for heavily trafficked, high-speed highways (6-ft inside shoulders plus 4 ft for a barrier and offset allowances). Side slopes are 2:1, and the break-even median widths are based on the cost curves shown in Figure 11. The unusual arrangement of break-even curves for the

several ground slopes results from the nonuniform variation in annual cost differences caused by the introduction of guardrail at specific ground slopes as shown in Figure 11. At a right-of-way cost of \$2,000/acre, breakdown median widths of 100 to 250 ft can be added to minimum median widths of 50 to 150 ft (Fig. 4) depending on the specific ground slope. Even at \$4,000/acre, independent alignment can provide total median widths ranging from about 100 to 200 ft at a total annual cost no greater than that of the narrow-median design with barrier.

The additional median width provided on independent alignments by both the economic break-even median width and the 100-ft safety median width requirement will have a slope flatter than the adjacent roadside slopes if the ground slope is uniform. The resulting shelf-like configuration would probably help to check the downward momentum of out-of-control vehicles from the upper roadway, the extent of speed reduction being dependent on its width and steepness. (Obviously a very narrow, unrounded "shelf" might present a severe overturning hazard to an out-of-control vehicle, and such designs should be avoided.) If that relief slope is ignored, the safety performance of the independent alignment design is underestimated somewhat, and safety comparisons with the narrow-median alternatives might thus be considered somewhat conservative.

In extremely flat terrain where roadways are built with minimal excavation or entirely on low fills the concept of the independent alignment no longer fully applies. However, the use of wide medians is still often substantially more economical than designs with narrow medians, without barriers as shown by the 0 percent curve in Figure 12. Roadways were assumed to be entirely on embankment with a fill height of 3 ft at the outside edge of the subgrade. Median fill slopes were taken as 6:1 to be consistent with current practice and the geometry shown in Figure 1. In this case the earthwork cost is somewhat greater for the wide-median design, and this extra cost plus the increased right-of-way cost is balanced against the cost of installing and maintaining a median barrier. The California Division of Highways standard of median width 40 ft or greater for barrier-free design is met for right-of-way cost up to \$4,000/acre (24-ft break-even median width plus the 16-ft comparison width).

APPLICATION CONSIDERATIONS

Certainly the uniform terrain conditions assumed in this study will seldom be found in practice, and actual annual costs will differ somewhat from those presented here. However, it is believed that "theoretical" break-even charts of the type presented here can still adequately reflect the relative economy of alternative median designs, particularly when applied to individual route analysis segments by using weighted average ground slopes. (Frequent changes in median design, however, as between independent-alignment and narrow-median designs with barrier, should be avoided regardless of the outcome of the economic analyses.)

Break-even charts based on 4-lane cross sections can be used for quick preliminary evaluation of designs having 6 or 8 lanes, if it is recognized that the increased number of lanes tends to favor independent alignment. On the other hand, designs with fore-slopes or ditches narrower than those used in the development of the charts will tend to favor narrow-median designs. Superelevation will tend to increase the cost of narrow-median designs more than that of independent-alignment designs.

Break-even charts can be readily developed for a wide range of cost and design conditions and used as a guide or starting point, followed where necessary by detailed analyses. The general procedure lends itself readily to computer treatment and can be incorporated in basic earthwork and route-alignment programs.

SUMMARY AND CONCLUSIONS

Highway planners and designers should give increased attention to the benefits and feasibility of independent alignment and other wide-median alternatives for median design. These designs are generally superior to narrow-median designs from the stand-point of aesthetics and traffic safety. For a wide range of typical cost and terrain conditions associated with many rural and suburban locations, the annual cost of independent-alignment designs will be little more, and in many instances considerably less, than the cost of alternative narrow-median designs, especially where a median

barrier is required with the latter. Independent alignments and other wide-median designs with median widths of up to 100 to 200 ft can often be built and maintained at no greater cost than some narrow-median designs, even where right-of-way cost is as high as \$4,000 to \$5,000/acre.

Economic comparison charts incorporating median and roadside safety requirements can be easily developed for quick preliminary evaluation of alternative median designs for representative design and cost conditions. The analysis procedure can be readily incorporated in routine earthwork computer programs.

REFERENCES

1. Neuzil, D., and Peet, J. S. Flat Embankment Slope Versus Guardrail: Comparative Economy and Safety. Highway Research Record 306, 1970, pp. 10-24.
2. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, Washington, D.C., 1965.
3. Stonex, K. A. Roadside Design for Safety. HRB Proc., Vol. 39, 1960, pp. 120-152
4. Michie, J. D., and Calcote, L. R. Location, Selection, and Maintenance of Highway Guardrails and Median Barriers. NCHRP Rept. 54, 1968.
5. Glennon, J. C., and Tamburri, T. N. Objective Criteria for Guardrail Installation. Highway Research Record 174, 1966, pp. 189-206.
6. Tunnard, C., and Pushkarev, B. Man-Made America: Chaos or Control? Yale Univ. Press, New Haven, Conn., 1963.
7. Snow, B.. ed. The Highway and the Landscape. Rutgers Univ. Press, New Brunswick, N.J., 1959.
8. Rapuano, M., et al. The Freeway in the City. U.S. Govt. Print. Office, 1968.
9. Highway Guardrail: Determination of Need and Geometric Requirements. HRB Spec. Rept. 81, 1964.

every 2 hours a minimum level of 25 percent of the time the vehicle is in motion is off road. This is a minimum value and is not based on any evidence or analysis.

HIGHWAY CURVE DESIGN FOR SAFE VEHICLE OPERATIONS

John C. Glennon and Graeme D. Weaver, Texas Transportation Institute,
Texas A&M University

Current design practice for horizontal curves assumes that vehicles follow the path of the highway curve with geometric exactness. The adequacy of this assumption was examined by conducting photographic field studies of vehicle maneuvers on highway curves. Results indicate that most vehicle paths, regardless of speed, exceed the degree of highway curve at some point on the curve. For example, on a 3-deg highway curve, 10 percent of the vehicles can be expected to exceed 4.3 deg. A new design approach is proposed. This approach is dependent on selecting an appropriate vehicle path percentile relation, a reasonable safety margin to account for unexplained variables that may either raise the lateral friction demand or lower the available skid resistance, and a minimum skid resistance versus speed relation that the highway department will provide on all pavements.

•SLIPPERY pavements have existed for many years; but the causes of slipperiness, its measurement, and its effect on traffic safety were not of great concern before 1950. Although reliable data on skidding accidents are hard to find, those in existence suggest that the skidding-accident rate has increased to proportions that may no longer be ignored. This trend may be due to improved accident reporting but also undoubtedly reflects increased vehicle speeds and traffic volumes (1).

More rapid accelerations, higher travel speeds, and faster decelerations made possible by modern highway and vehicle design have raised the frictional demands on the tire-pavement interface. Larger forces are required to keep the vehicle on its intended path. On the other hand, when pavements are wet the frictional capability of the tire-pavement interface decreases with increasing speed. In addition, higher traffic volumes and speeds promote a faster degradation in the frictional capability of the pavement.

From the technological standpoint, the slipperiness problem appears amenable to solutions that either reduce the frictional demand (improved geometric design and reduced speed limits for wet conditions) or increase the frictional capability (improved pavement surface design, tire design, and vehicle inspection procedures). This research study was concerned with the adequacy of geometric design standards for horizontal curves.

A previous report (2) indicated that current standards (3) for minimum horizontal curve design may not give an adequate factor of safety for modern highway operations. Evaluation of the state of the art revealed several uncertain features of the design basis. The adequacy of the following 4 assumptions was questioned:

1. Vehicles follow the path of a highway curve with geometric exactness;
2. The point-mass equation, $e + f = V^2/15R$, defines the impending skid condition;
3. Lateral skid resistance can be measured with a locked-wheel skid trailer; and
4. Levels of lateral acceleration that produce impending driver discomfort can be used for design values to ensure an adequate factor of safety against lateral skidding.

The goal of this current research was to perform field studies, simulation studies, and controlled experiments to test assumptions 1, 2, and 3. With objective data from that research, the adequacy of assumption 4 would then be evaluated.

The simulation studies and controlled experiments to test assumptions 2 and 3 were done on another project conducted at the Texas Transportation Institute (4). The major emphasis of the research reported here, therefore, was to empirically relate vehicle paths to highway curve paths to test assumption 1. Then, by evaluating the data generated by the 2 research studies, revised curve design standards could be proposed, if appropriate.

FIELD PROCEDURES

The general procedure was to record vehicle paths on movie film by using a camera housed in a following vehicle. That observation vehicle, stationed beside the highway about 1 mile upstream from a highway curve site, was driven onto the highway behind a subject vehicle as it passed. The observation vehicle was then accelerated to close the position headway so that the subject vehicle was within 60 to 100 ft as it approached the curve site. The vehicle path was filmed continuously from about 150 ft upstream to 150 ft downstream of the highway curve.

Study Sites

Five highway curve sites, ranging in curvature from 2 to 7 deg, were selected within a 30-mile radius of College Station, Texas. All curve sites were in rural areas and had essentially level vertical curvature. None of the curve sites had spiral transitions; that is, they were all joined by tangent alignment at both ends of the circular curve. Super-elevation rates ranged from 0.04 to 0.08.

Equipment

A 1970 Ford half-ton pickup truck was used as the observation vehicle. So that subject drivers would be unaware of being photographed, the camera and operator were concealed in a box mounted on the truck bed. The box, resembling a tool shed, was directly behind the truck cab, standing 24 in. above the cab roofline. The observation vehicle is shown in Figure 1.

Subject vehicles were photographed through a small window over the left side of the cab. It is doubtful that subject drivers were aware of being photographed because the window was the only opening; therefore, the box appeared dark and unoccupied.

An Arriflex 16-mm camera was used to photograph curve maneuvers. Power was supplied by an 8-volt battery through a governor-controlled motor to produce a constant 24 frame/sec film advance. The film was black and white Plus-X reversal (Kodak, ASA 50) on 400-ft reels.

Subject vehicles were photographed with a zoom lens (17.5 to 70.0 mm) so that the cameraman could maintain field of view and, at the same time, obtain the largest possible view of the left-rear tire of the vehicle. The camera was mounted on a ball-head rigid base attached to a shelf. The camera and mounting configuration are shown in Figure 2.

Geometric Reference Marks

The plan was to measure the lateral placement of the subject vehicle's left-rear tire at intervals along the highway curve by using the geometric centerline of the highway curve as a base reference. Two-foot lengths of 6-in. wide temporary traffic line pavement markings were placed perpendicular to, and centered on, the centerline at 20-ft intervals throughout each study site. The 2-ft markers gave a length calibration that was always pictured on the film frame where lateral placement measurements were taken. The 20-ft interval gave a reference system for speed and radius calculations.

Sampling Procedures

About 100 vehicles were sampled for each curve site. This number has no statistical basis but was set by time and monetary constraints for data collection and film analysis. About half of the samples were taken for each direction of traffic at each curve site. Samples were limited to passenger cars and pickup trucks.

After each photographic sample was taken, the observation vehicle returned to its roadside position at the starting station, about 1 mile upstream from the curve site. The next sample was the first free-flowing vehicle that passed the starting station and had enough clear distance to the rear to allow the observation vehicle to move in behind. This procedure allowed for an essentially random selection of sample speeds.

FILM ANALYSIS

The film was analyzed with a Vanguard motion analyzer. That device is a portable film reader for measuring displacements on photographic projections. It consists of a projection head, projection case, and measurement screen.

The 16-mm projection head permits forward and reverse motion of film on 400-ft reels. A variable-speed mechanism moves the image across the projection screen from 0 to 30 frames/sec. A counter on the projection head displays frame numbers. If the camera speed is known, then, by noting elapsed frames, displacement over time (speed) can be calculated.

The measurement screen has an X-Y cross-hair system that measures displacement in 0.001-in. increments on the projected image. Rotation of the measurement screen permits angular alignment of the cross hairs with the projected image. Two counters display the numerical positions of the movable cross hairs. Conversion of image measurements to real measurements requires a calibration mark of known length in the plane of the photographed object. In other words, the 2-ft markers used at the highway curve sites were measured in machine units on the film image to give a calibration for converting image length to real length.

In the analysis of the curve maneuver samples, the lateral vehicle position reference was always the left edge of the left-rear tire. Lateral placement at each reference marker was measured from the frame where the left-rear tire was nearest the marker. After calibration readings on the left and right edge of the reference marker were recorded, the position reading of the left-rear tire was recorded. These readings, along with the 2-ft known length, gave the data necessary for calculating the actual lateral placement.

MATHEMATICAL ANALYSIS

The Vanguard data were used in a computer program to calculate vehicle speed, left-rear tire lateral placement, vehicle path radius, and lateral friction demand f . Those estimates were calculated for each sample at each reference marker.

Vehicle Speed

The estimate of vehicle speed at each reference marker was obtained as the average speed over a distance interval. Selection of the interval was dependent on the error sensitivity from 2 sources. The smaller the interval was, the smaller was the error due to sudden speed changes and the greater was the error due to integer frame-count estimates (number of frames elapsed as the vehicle traveled the 20 ft between successive markers). Because the samples did not exhibit large speed changes over short intervals, the accuracy of the instantaneous speed was most sensitive to the frame-count estimate.

Because the frame-count estimate was to the nearest integer, the greatest frame-count error at a point was $\frac{1}{2}$ frame. For an analysis length, the greatest error in frame-count difference was 1 frame ($\frac{1}{2}$ frame at each end). Figure 3 shows the sensitivity of the speed estimate to frame-count differences for several analysis intervals. To reasonably diminish this error source, we set the speed estimate analysis interval at 160 ft. Therefore, the speed estimate at each reference marker was the average speed over the 160-ft interval centered on that marker.

Vehicle Radius

The computer program calculated the lateral placement of the left edge of the left-rear tire at each reference marker. The instantaneous vehicle path radius was then

Figure 1. Observation vehicle.



Figure 2. Camera and mounting.

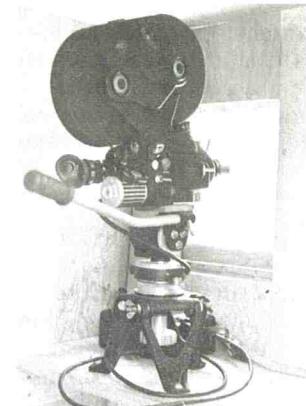


Figure 3. Sensitivity of speed estimate to frame count.

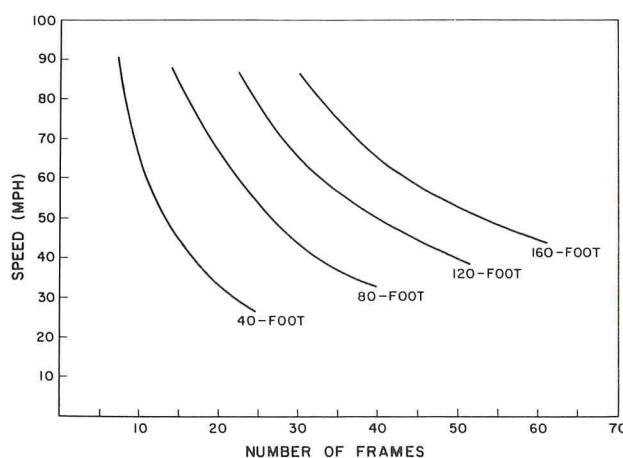


Figure 4. Geometric description of vehicle radius calculation.

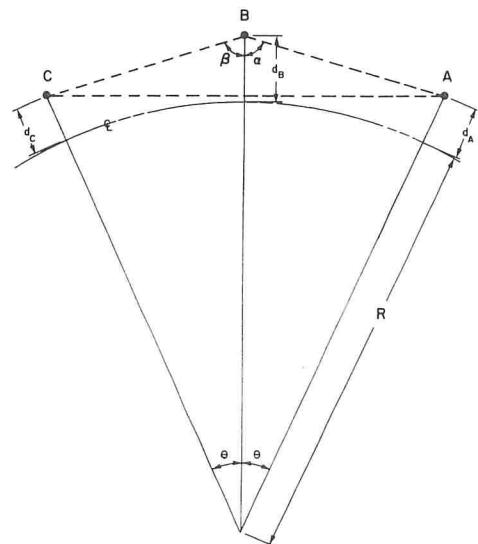
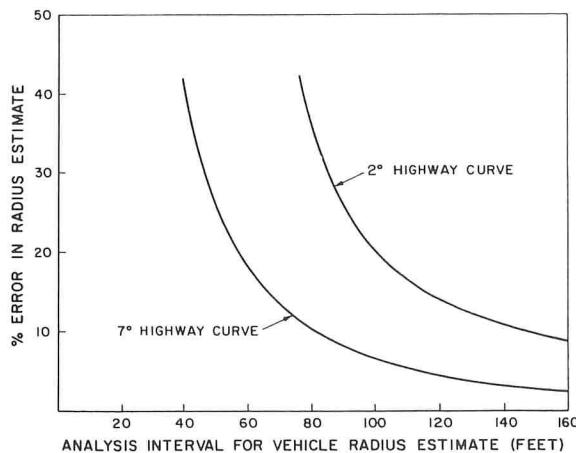


Figure 5. Error sensitivity of vehicle radius estimate.



estimated by computing the radius of the circular curve through 3 successive tire positions, the center position being at the reference marker under consideration. Inasmuch as a circular arc is the minimum curved path through 3 points, the radius so calculated is a conservative estimate of the smallest instantaneous radius over the interval.

Figure 4 shows the geometric description of the vehicle radius calculation. Points A, B, and C represent left-rear positions at equal intervals along the highway curve. The estimated vehicle path radius R_v is the radius of the circular arc that circumscribes triangle ABC. The following calculations were performed to obtain that radius.

From the law of cosines, lines AB, BC, and AC are (in ft)

$$\overline{AB} = \sqrt{(R + d_A)^2 + (R + d_B)^2 - 2(R + d_A)(R + d_B) \cos \theta}$$

$$\overline{BC} = \sqrt{(R + d_B)^2 + (R + d_C)^2 - 2(R + d_B)(R + d_C) \cos \theta}$$

$$\overline{AC} = \sqrt{(R + d_A)^2 + (R + d_C)^2 - 2(R + d_A)(R + d_C) \cos 2\theta}$$

where

d_A , d_B , and d_C = lateral displacements from the centerline at points A, B, and C, ft;

R = radius of the highway curve, ft; and

θ = central angle subtended by arc length of half the analysis interval.

From the law of sines,

$$\alpha = \sin^{-1} [(R + d_A)(\sin \theta)/\overline{AB}]$$

$$\beta = \sin^{-1} [(R + d_C)(\sin \theta)/\overline{BC}]$$

The radius of the vehicle path R_v that circumscribes triangle ABC is then calculated by

$$R_v = \overline{AC}/2 \sin(\alpha + \beta)$$

As with the speed estimate, it is necessary to look at the error sensitivity of the radius estimate for various analysis intervals. Any error in the radius estimate would, of course, come from an error in the lateral placement estimate. Although study control was exerted, small errors were possible from several sources, including (a) lateral discrepancy in placing the reference marker, (b) length discrepancy of the reference marker, (c) film parallax, (d) sampling error due to taking lateral placement readings up to $1/2$ frame away from the reference marker, (e) equipment error, and (f) human error in reading and recording lateral placement measurements.

Estimating the distribution of error values for lateral placement estimates was not possible. All the error sources could be either positive or negative, and some error cancelation normally would be expected. In addition, all error sources would not be expected to reach maximum in the same direction at the same time.

An error of 0.10 ft in the lateral placement estimate was assumed to check the error sensitivity of the radius estimate for various analysis intervals. For this analysis, the correct path was assumed to be the path of the highway curve. Therefore, the error has the effect of changing the middle ordinate M of the circular arc. The middle ordinate M of the correct circular arc and the middle ordinate M_e of the circular arc in error are as follows (in ft):

$$M = C/2 \tan DC/400$$

$$M_e = M + 0.10 = C/2 \tan D_e C/400$$

where

- C = chord length (approximately by arc length over short intervals) of both curves, ft;
- D = deg of correct path; and
- D_e = deg of path in error.

If d is the absolute error in curve degree, then

$$d = D_e - D$$

Solving for D and D_e in the first 2 equations, we obtain

$$d = 400/C \{ \tan^{-1} [2(M + 0.10)/C] \} - 400/C[\tan^{-1}(2M/C)]$$

or

$$d = 400/C (\tan^{-1} 1/5C)$$

if E is the percentage of error of the vehicle path degree estimate (and likewise the percentage of error of the radius estimate), then

$$E = 100 \{ [400/C (\tan^{-1} 1/5C)]/D \}$$

Figure 5 shows the percentage of error of the radius estimate for an absolute lateral placement error of 0.10 ft. The percentage of error is plotted against the length of analysis interval for the range in highway curves studied. The figure shows that the error sensitivity is greatly reduced as the analysis interval is increased.

For calculating instantaneous radius estimates, the analysis interval was set at 160 ft; a greater interval would increase the chance of grossly overestimating the smallest instantaneous radius by diluting the true path deviations at the 20-ft intervals.

Lateral Friction Demand

The lateral friction demand at the tire-pavement interface was estimated at each reference marker for each sample by using the centripetal force equation, $f = V^2/15R - e$. The results of full-scale vehicle skidding tests (4) on several pavements indicated that this equation is a reasonably good predictive tool.

RESULTS

The result of the computer application was the printing of 50 to 80 data sets of lateral placement, speed, instantaneous radius, and lateral friction demand for each vehicle sampled. The critical point of each sample was represented by selecting the point of maximum friction demand. This point, for a great number of samples, coincided with either the point of maximum speed or the point of minimum path radius or both. Sample data, showing the ranges in speed, vehicle radius, and lateral friction demand, are given in Table 1 for each study site.

When the design equation $e + f = V^2/15R$ (in which R is the vehicle path radius) was used, the highway curve radius we assumed to be equal to the vehicle path radius. The study data showed that this assumption is invalid. The problem for design then is to write the equation not in terms of vehicle path radius but in terms of highway curve radius. Thus, a representative form of the vehicle path radius in terms of the highway curve radius is called for.

The original research plan was to generate for each highway curve a relation between vehicle path radius and vehicle speed at the point of maximum lateral friction demand. With this information, an acceptable highway curve radius could be designed for any combination of design speed V , design lateral friction demand f , and superelevation rate e .

Plotting scatter diagrams of speed versus radius did not indicate any relation between the 2 parameters. This fact was verified by conducting a simple linear regression analysis on the data. For the 5 highway curves studied, the vehicle speed explained no more

Table 1. Speed, vehicle radius, and lateral friction demand.

Site and Curve	Sample	Left or Right	Initial Speed	Maximum Speed	Minimum Radius	Data at Point of Maximum f			
						Quarter of Curve	Speed	Radius	f
1, 7 deg	105	L	61	69	732	4	67	769	0.356
	22	L	60	60	647	1	56	647	0.285
	84	L	60	61	806	2	58	811	0.243
	11	R	47	51	690	4	51	700	0.201
	104	R	45	45	587	1	43	587	0.159
	91	R	42	42	718	1	41	735	0.102
3, 5 deg	37	L	75	75	909	1	75	909	0.340
	30	L	65	69	996	1	69	1,087	0.271
	58	R	64	67	973	1	67	1,113	0.240
	53	L	56	57	780	4	55	780	0.184
	25	R	56	56	995	1	55	1,205	0.135
	49	L	42	43	1,000	1	43	1,044	0.098
2, 4 deg	101	R	77	77	1,003	4	75	1,042	0.295
	105	R	65	65	913	4	65	913	0.228
	89	L	54	61	750	3	56	787	0.188
	5	L	57	58	967	1	58	967	0.158
	79	L	58	64	1,267	4	64	1,354	0.126
	37	R	41	41	1,239	4	40	1,239	0.002
5, 2.5 deg	75	L	73	75	1,582	3	75	1,582	0.186
	18	R	69	69	1,022	4	61	1,022	0.182
	7	R	69	71	1,662	2	69	1,662	0.130
	43	L	67	69	1,834	3	67	1,834	0.114
	97	L	62	64	1,974	4	64	2,045	0.083
	98	R	46	49	1,499	4	48	1,499	0.041
4, 2 deg	99	R	73	75	1,694	3	75	1,715	0.177
	92	R	69	69	2,084	4	69	2,084	0.132
	52	R	67	67	1,865	3	65	1,865	0.113
	24	R	60	60	1,495	1	57	1,495	0.084
	73	L	64	64	2,117	3	62	2,365	0.080
	40	L	44	44	1,536	4	38	1,536	0.032

Figure 6. Percentile distribution of vehicle path radius versus highway radius.

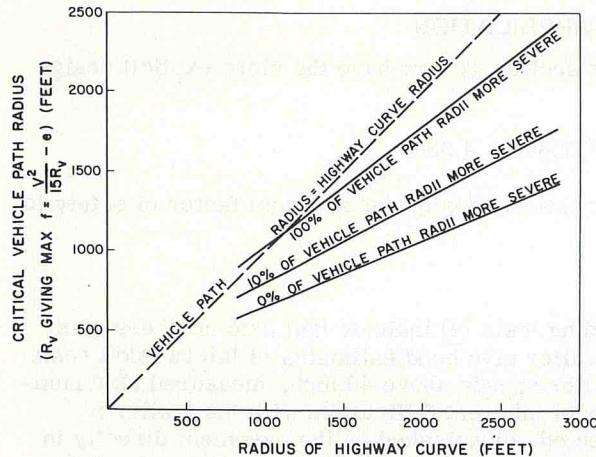
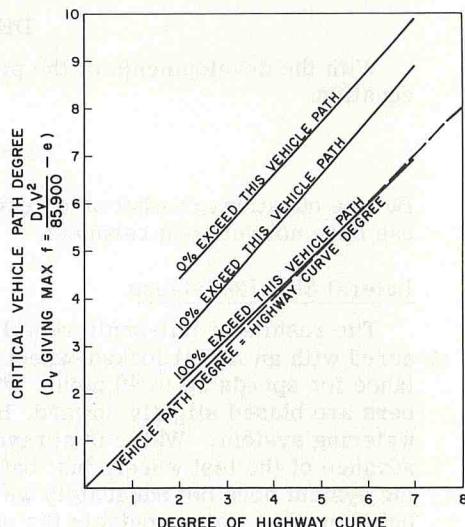


Figure 7. Percentile distribution of vehicle path degree versus highway curve degree.



than 11.4 percent ($r^2 = 0.114$) of the variation in the vehicle path radius, and in fact, for 3 of the study sites, explained than 4.7 percent.

Actually, the lack of correlation between vehicle radius and speed simplified the analysis, because it indicated that the distribution of vehicle path radii (at maximum lateral friction demand) found for each site could be expected at any speed within the speed range studied. Therefore, percentiles of vehicle path radius could be plotted for each highway curve radius.

Figure 6 shows simple linear regression fits for different percentiles of vehicle path radius versus highway curve radius. Figure 7 shows a similar relation in terms of degrees of curve. For these 2 graphs, the equations of the lines and their goodness of fit are given in Table 2. What these relations show, for example, is that, on a 3-deg highway curve, 10 percent of the vehicles will exceed a 4.3-deg path maneuver.

To arrive at the design relation for highway curve radius, a percentile level is needed that ensures that very few vehicles will approach instability. The 10 percent level appears to be a reasonable choice. Using this level for design would say that only 10 percent of the vehicles traveling at design speed will exceed a given vehicle path radius. This level would change the design equation, in terms of highway curve radius R , to read

$$e + f = V^2/7.86R + 4,030$$

or

$$e + f = (D + 0.9) V^2/76,100$$

It is interesting to analyze these equations to see what percentage of vehicles might exceed the design f . For example, if we design a curve for 60 mph with 0.06 super-elevation and a design $f = 0.13$, what percentage of vehicles will exceed the design f ? For the given design parameters, using the modified design equations gives $R = 1,890$ ft and $D = 3.1$ deg. Using the equations given in Table 2, it can be found that design f will be exceeded by 0 percent of the vehicles at 53 mph, 10 percent at 60 mph, 50 percent at 64 mph, and 100 percent at 70 mph.

DESIGN APPLICATION

With the developments of the previous section we now have the more explicit design equation:

$$e + f = V^2/7.86R + 4,030$$

But the questions of what skid resistance level to design for and what factor of safety to use have not yet been resolved.

Lateral Skid Resistance

The results of full-scale vehicle skidding tests (4) indicate that skid numbers measured with an ASTM locked-wheel skid trailer give good estimates of lateral skid resistance for speeds up to 40 mph. With trailer speeds above 40 mph, measured skid numbers are biased slightly upward, because of inherent difficulties with the trailer's watering system. Water is sprayed, ejected, or splashed on the pavement directly in advance of the test wheel, just before and during lockup. At higher speeds, the watering system does not adequately wet the pavement because of the small time difference between when water contacts the pavement and when the tire is skidded over the wetted segment.

Standard skid trailer tests compared with tests made with an external watering source, for a speed range of 20 mph to 60 mph, revealed that measurements at speeds above 40 mph gave lower skid resistance values for the external watering tests. In addition, using the skid number versus speed relation for external watering in the centripetal force equation more closely predicted the results of full-scale spin-out tests conducted on the project. The results are inconclusive, however, because only 2 pavements were used.

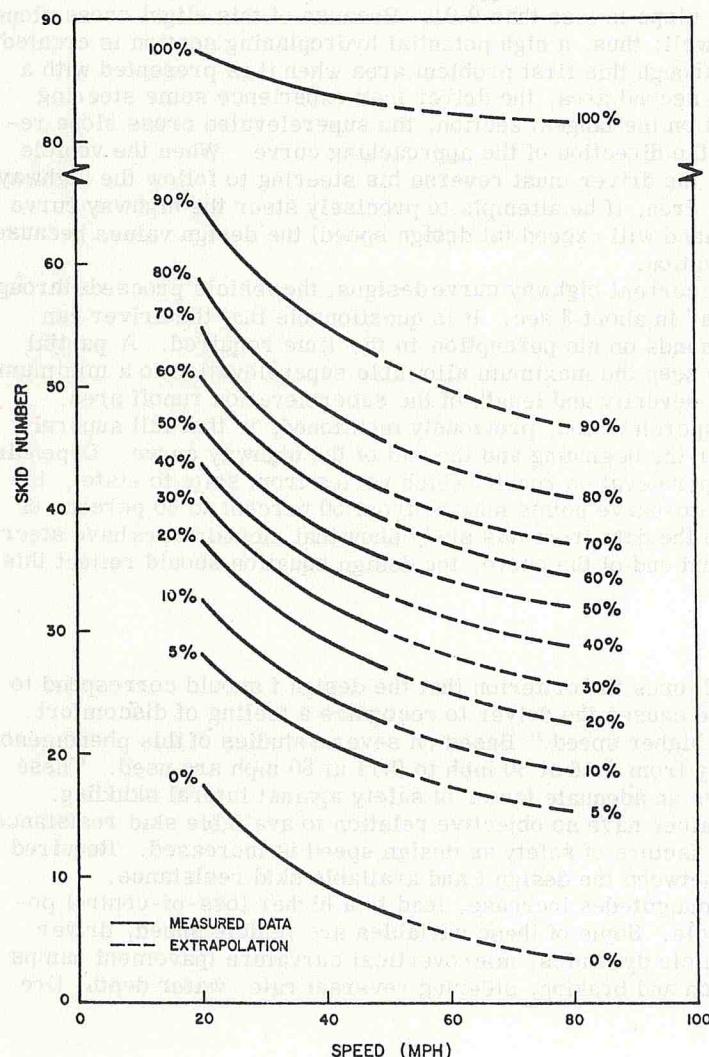
Table 2. Percentiles of vehicle path versus highway curve path.

Vehicle Path Degree Greater Than D_v (percent)	Vehicle Path Radii Lower Than R_v (percent)	Equation	Goodness of Linear Fit (r^2)
0		$D_v = 2.427 + 1.057 D$	0.930
5		$D_v = 0.984 + 1.165 D$	0.985
10		$D_v = 1.014 + 1.128 D$	0.984
15		$D_v = 0.894 + 1.124 D$	0.983
50		$D_v = 0.796 + 1.030 D$	0.991
100		$D_v = 0.474 + 0.919 D$	0.986
0	0	$R_v = 225.1 + 0.416 R$	0.990
5	5	$R_v = 266.0 + 0.510 R$	0.953
10	10	$R_v = 268.0 + 0.524 R$	0.951
15	15	$R_v = 271.1 + 0.538 R$	0.956
50	50	$R_v = 267.5 + 0.611 R$	0.971
100	100	$R_v = 276.7 + 0.751 R$	0.987

Table 3. Hypothetical solution of proposed design equation where $e = 0.06$ and $M_s = 0.10$.

SN ₄₀	Design Speed (mph)	Design Radius (ft)	Design Value (deg)
35	50	640	9.0
	60	1,270	4.5
	70	2,080	2.8
	80	3,000	1.9
	25	560	10.2
	50	1,400	4.1
	60	2,500	2.3
	70	3,900	1.5
	80	5,700	1.0

Figure 8. Percentile distribution of skid number versus speed relation for 500 pavements in 1 state.



Other research (5) using 15 pavement surfaces showed similar comparisons of the skid number versus speed relation for internal and external watering tests. The measured differences, however, were not too substantial, amounting to an average of 0.04 at 60 mph. That difference varies between pavements; therefore, it is difficult to predict. Perhaps the difference can best be accounted for in the design process by providing an adequate safety margin between predicted friction demand and measured skid number.

Spiral Transitions

Although no specific research was done to study spiral transitions for highway curves (all sites had tangent to circular curve transitions), the data do indicate that spiral transitions may be desirable. The data clearly revealed that many drivers have trouble transitioning their vehicle path from tangent to circular curve and from circular curve to tangent. This fact is shown by the majority of samples that had their highest lateral friction demand in either the first quarter or the last quarter of the highway curve.

Superelevation

A previous report (2) has shown that the higher the superelevation is the greater the problem is of driving through the area of superelevation runoff for unspiraled highway curves that curve to the left. As the vehicle approaches the curve, it is presented first with an area where the cross slope is less than 0.01. Because of this slight cross slope, the pavement does not drain well; thus, a high potential hydroplaning section is created. The vehicle no sooner gets through this first problem area when it is presented with a second problem area. In the second area, the driver may experience some steering difficulty because, while still on the tangent section, the superelevated cross slope requires him to steer opposite the direction of the approaching curve. When the vehicle gets to the point of tangency, the driver must reverse his steering to follow the highway curve. In this third problem area, if he attempts to precisely steer the highway curve path, the lateral friction demand will exceed (at design speed) the design values because this area lacks full superelevation.

At design speed, for most current highway curve designs, the vehicle proceeds through this "compound dilemma area" in about 3 sec. It is questionable that the driver can adequately react to those demands on his perception in the time required. A partial solution to this problem is to keep the maximum allowable superelevation to a minimum. This practice will reduce the severity and length of the superelevation runoff area.

The other problem with superelevation, previously mentioned, is that full superelevation is not available near the beginning and the end of the highway curve. Depending on the design practice for superelevation runoff (which varies from state to state), the superelevation at the tangent-to-curve points may be from 50 percent to 80 percent of full superelevation. Because the data from this study show that most drivers have steering trouble at the beginning and end of the curve, the design equation should reflect this reduced superelevation.

Safety Margin

Current design practice (3) uses the criterion that the design f should correspond to "that point at which side force causes the driver to recognize a feeling of discomfort and act instinctively to avoid higher speed." Based on several studies of this phenomenon, design values ranging linearly from 0.16 at 30 mph to 0.11 at 80 mph are used. These values are assumed (3) to give an adequate factor of safety against lateral skidding.

Actually, these design f values have no objective relation to available skid resistance. In addition, they give smaller factors of safety as design speed is increased. Required is a more realistic relation between the design f and available skid resistance.

Many variables, as their magnitudes increase, lead to a higher loss-of-control potential for the cornering vehicle. Some of these variables are vehicle speed, driver steering judgment, faulty vehicle dynamics, microvertical curvature (pavement bumps and dips), vehicle acceleration and braking, steering reversal rate, water depth, tire

temperature, tire wear, and wind gusts. On wet pavements, vehicle speed is the most significant variable, not only because the lateral friction demand increases with the square of the speed but also because skid resistance decreases with speed. These 2 phenomena, of course, are already accounted for in the design procedure. Also, with the design equation modified to account for statistical values of vehicle path, the effects of driver steering judgment and faulty vehicle dynamics are probably taken care of. Therefore, factors of safety are only needed to give some margin of error for the remaining variables.

Because these other variables have not been explicitly evaluated, it is difficult to determine representative factors of safety. It seems clear, however, because vehicle paths are now explicit in the design equation, that lower factors of safety can be used. In addition, there may be a low probability of the other variables causing a considerably greater lateral friction demand than already accounted for in the modified design equation.

Because skid resistance varies by pavement, a safety margin is a better tool than a factor of safety. A constant safety margin also has the advantage of giving a higher factor of safety as design speed is increased. Although there are no supporting data, a safety margin in the range of 0.08 to 0.12 should reasonably allow for the unaccounted variables, including the deviation between actual and measured skid numbers.

Design Equation

With all the considerations previously discussed, it is possible to modify the design equation into a comprehensive tool. The original equation is

$$e + f = V^2 / 15R_v$$

If the derived expression for the tenth percentile R_v in terms of the highway curve radius R is substituted, the following equation results:

$$e + f = V^2 / 7.86R + 4,030$$

If the reduced superelevation at the beginning and end of the curve is approximated by $0.7e$, and if f is expressed by the skid number SN_v , divided by 100, minus a safety margin M_s , the following equation results:

$$0.7e + SN_v/100 - M_s = V^2 / 7.86R + 4,030$$

or

$$R = -514 + V^2 / [5.48e + 7.86 (0.01 SN_v - M_s)]$$

It is not possible, of course, to use this equation for design unless, first, a safety margin is selected and, second, a "typical" skid resistance versus speed relation is selected. The latter makes it difficult to give specific recommendations for design standards. Essentially, the selection of a skid resistance versus speed relation depends on what minimum level of skid resistance the highway department provides.

Hypothetical Design Use

Although specific design standards cannot be recommended, the sensitivity of the suggested design equation to minimum skid resistance levels is important. Figure 8, a percentile distribution of skid numbers in one state, will be used for illustration. The 2 curves having skid numbers of 35 and 25 at 40 mph will be used as hypothetical minimum skid resistance requirements. The value of 35 has been widely recommended.

A safety margin of 0.10 and a design e of 0.06 will yield solutions to the design equation for the various skid resistance levels as given in Table 3. The design values below 2 deg are somewhat questionable because of the limits of the field data. It can be shown, though, that a 2-deg curve will not satisfy the selected safety margin for these higher design speeds.

REFERENCES

1. Kummer, H. W., and Meyer, W. E. Tentative Skid-Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967.
2. Glennon, J. C. State of the Art Related to Safety Criteria for Highway Curve Design. Texas Transportation Institute, Res. Rept. 134-4, Nov. 1969.
3. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, 1965.
4. Ivey, D. L., Ross, H. E., Hayes, G. G., Young, R. D., and Glennon, J. C. Side Friction Factors Used in the Design of Highway Curve. Texas Transportation Institute, unpublished rept., March 1971.
5. Gallaway, B. M., and Rose, J. R. Highway Friction Measurements With Mu-Meter and Locked Wheel Trailer. Texas Transportation Institute, Res. Rept. 138-3, June 1970.

COST-EFFECTIVENESS TECHNIQUE FOR ANALYSIS OF ALTERNATIVE INTERCHANGE DESIGN CONFIGURATIONS

Joseph A. Wattleworth, Department of Civil Engineering, University of Florida; and Jerry W. Ingram, Florida Department of Transportation

•AT THE present time highway designers generally have inadequate means for selecting the most cost-effective configuration for a given set of traffic volumes. Frequently the designer simply seeks out a configuration that meets the requirements and selects that one. There is a need for a procedure that can be quickly used by a designer to compose alternative interchange design (or redesign) configurations and that considers the cost of each configuration as well as the effectiveness of the interchange. In addition, it would be highly desirable if the procedure would allow the designer to use the results of the analysis of one configuration to lead him to the next logical configuration, thereby giving him the capability of a sequential design process.

This report develops a cost-effectiveness methodology for the analysis and comparison of alternative interchange configurations. The methodology is presented in the framework of a case study of the interchange of Fla-436 and I-4 north of Orlando, Florida.

CHARACTERISTICS OF THE INTERCHANGE

At present this is a conventional diamond interchange. Interstate 4 is a 4-lane divided freeway, and Fla-436 is also a 4-lane divided arterial street in a semirural area. All interchange ramps are 1-lane, and an inadequate signal system is in operation. The interchange is currently overloaded during the morning and afternoon peak periods, and a high traffic growth rate (approximately 8 percent yearly) is expected. One major variable in the traffic projections is the possible construction of the proposed Beltline Expressway that will essentially parallel Fla-436. Therefore, traffic projections were made under 2 assumptions: that the Beltline Expressway was built and that it was not built. The inability to predict a single set of future design volumes was one of the factors that led to the development of the cost-effectiveness methodology.

STUDY PROCEDURE

The basic procedure that was utilized in this study is a general systems approach. Described briefly, the initial step is to define the objectives of the study and to analyze the alternative systems that might satisfy those objectives. The general objective of this particular analysis is to select the geometric design-operations alternative for the interchange of I-4 and Fla-436 that will provide the required traffic capacity at minimum cost. All alternative solutions, referred to as candidate systems, are compared to determine which alternative will best meet the existing traffic demands and also sustain future demands and which will do so at a minimum cost. Comparison of alternatives was done on the basis of a cost-effectiveness analysis, where effectiveness was defined in terms of the total interchange capacity (expressed as equivalent ADT entering the interchange). The most important characteristic of a systems approach is that it in-

volves the consideration of all alternatives and their analyses and comparison to determine the most effective or optimal candidate system.

Model to Determine Effectiveness of Each Candidate System

Each geometric configuration alternative was evaluated in terms of the total peak-hour capacity, that is, vehicles/hour. There are 2 reasons for using peak-hour analysis rather than 24-hour capacity analysis:

1. Peak-hour capacity must be adequate in order for the interchange to operate efficiently during these periods; and
2. The afternoon peak-period traffic pattern is generally substantially different from the morning peak pattern.

A linear programming model, which had been developed earlier (1), was used to establish the peak-hour capacity of each alternative. Basically, this model solves for the peak-hour interchange capacity of any configuration subject to 2 types of constraints:

1. The volume using any interchange element does not exceed the capacity of the element; and
2. The traffic entering the interchange is properly distributed among all of the possible movements.

The capacity is the maximum peak-hour volume that can enter the interchange.

Utility of Interchange Analysis Model

The model determines the maximum peak-hour (entering) volume a particular interchange configuration can accommodate before congestion develops at a critical location within the interchange. The model also identifies the critical element that limits the total interchange capacity. With this model, the highway engineer can then select a new design configuration that provides an increased capacity for the critical element; and, thus, the new configuration will have a greater capacity. The model, then, can be used to determine the appropriate sequence of geometric improvements and the capacity of each alternative interchange configuration. Thus, the designers who use this model can determine a set of geometric improvements that will increase the capacity of an interchange and the order or relative priority that should be given to each improvement.

The steps involved in the use of the model for the analysis of an interchange are as follows:

1. Identify all reasonable types of interchange configurations, such as diamond, cloverleaf, partial cloverleaf, and directional, that are to be considered as possible solutions (this step is a function of the designer's general knowledge of the interchange area);
2. Set up the interchange analysis model for a first trial interchange configuration of one of these types (this may be the existing interchange configuration);
3. Solve the model to determine the total capacity of that interchange configuration and the critical interchange elements that limit the total capacity, e.g., those elements on which the volume equals the element's capacity;
4. Revise the design configuration of the interchange to provide increased capacity for the elements that are found to be critical.
5. Repeat the steps of modeling a new interchange configuration, solving the model to determine the critical elements, revising the design configuration, modeling the new configuration, and so on until an interchange configuration of adequate capacity is found or until no further improvements are feasible; and
6. Repeat steps 2 through 5 for each appropriate type of interchange configuration.

This step-by-step analysis ensures that for a given type of interchange each configuration considered has a higher capacity than the configuration previously considered. It also ensures that, at each step, the capacity of the critical interchange element is increased, and it allows the designer to focus his attention on the critical elements.

CANDIDATE SYSTEMS

Five alternate interchange configurations or candidate systems were analyzed to determine the maximum capacity of each during both the morning and afternoon peak hours; many of the systems had several variations included in the analyses.

The following section describes the candidate systems and discusses the geometric improvements in proceeding from one candidate system to another. The final variation of candidate systems II, III, IV, and V involves the modeling of Interstate 4 as a 6-lane freeway (at present, it has 4 lanes). This occurs at the step after the capacity of one or both of the 2-lane directional sections of I-4 becomes the critical interchange element.

The analysis of one configuration of one particular candidate system provides a capacity value for the configuration and identifies the interchange element that limits the capacity of the interchange. This, then, allows the designer to select the next configuration that provides a higher capacity for the critical element, and, hence, each configuration has a higher capacity than the preceding one.

CASE STUDY: I-4 AND FLA-436 INTERCHANGE

This section presents the case study that was conducted on the interchange of I-4 and Fla-436 north of Orlando, Florida. Six general candidate systems were considered, and the analysis included 17 specific candidate systems. Within each type of candidate system, the analysis of one candidate system led to the selection of the next until all feasible improvements had been made.

Capacity Analyses

For each of the 17 systems, a morning peak-hour capacity (total volume entering the interchange) and an afternoon peak-hour capacity analysis was conducted. It was assumed that the volume during each peak hour was 10 percent of the total volume entering the interchange. Thus, it was possible to determine an equivalent ADT by simply multiplying the lower of the 2 peak-hour capacities by 10. The equivalent ADT, which is also in terms of vehicles entering the interchange, was used as the measure of effectiveness of each interchange configuration (candidate system). The equivalent ADT, then, is the total daily traffic that can enter the interchange without causing any of the capacities in the interchange to be exceeded in either of the peak periods. Figure 1 shows the geometric configurations, and Table 1 gives a description of each system and the results of the capacity analysis for each.

Candidate System I—Candidate system I represents a slight improvement over the existing configuration and operation of the interchange of I-4 and Fla-436. In candidate system I, new signalization is considered so that the 2 closely spaced signals at the ramp terminals can be coordinated. This will also permit the use of the 4-phase, 2-overlap signalization that has been found to be highly efficient for diamond interchange signalization (2). In candidate system I, no reconstruction of interchange elements is considered.

Under this type of operation, the capacity during the morning peak hour is 5,991 vph, and the critical capacity element is the entrance ramp in the southwest quadrant. That is, for the given volume distribution in the morning peak period, the volume using the entrance ramp in the southwest quadrant exactly equaled its capacity, while the volume on each of the other geometric elements was less than its capacity. In the afternoon peak period the capacity was found to be 5,418 vph, and the critical capacity element was the exit ramp in the southeast quadrant. Because the afternoon capacity is lower than the morning capacity, the equivalent ADT is 54,180 vehicles, and the geometric element that restricts this value is the exit ramp in the southeast quadrant.

Candidate System II—Several relatively minor modifications of the existing geometric design configuration were considered. In each of the 4 variations of candidate system II, the improved signalization of candidate system I is assumed to be installed and in operation.

In the morning peak period, the east-to-south left-turn movement is very heavy and merges with the heavy west-to-south right-turn movement on the southbound entrance

Figure 1. Geometric configurations of candidate systems.

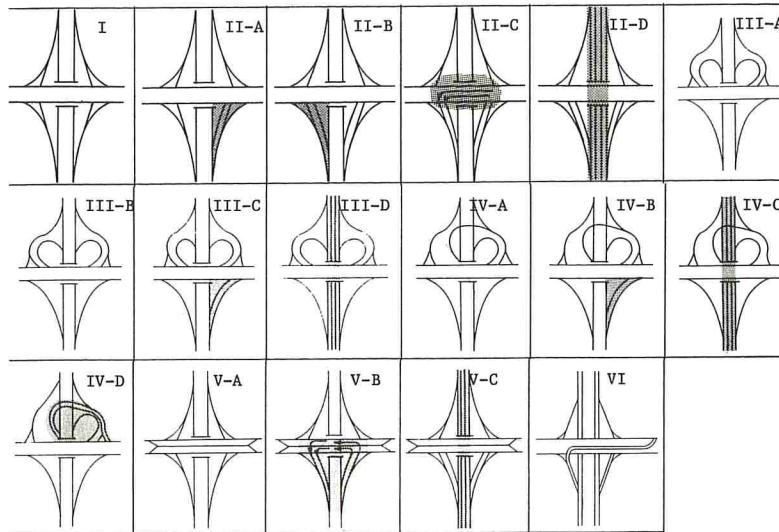


Table 1. Description of candidate systems and summary of traffic analyses.

System	Description	a. m.		p. m.		Equivalent ADT (entering)
		Peak-Hour Capacity	Critical Element	Peak-Hour Capacity	Critical Element	
I	New signalization to develop 4 phase-2 phase overlap	5,991	Entrance ramp in S. W. quadrant overloaded	5,418	Exit ramp in S. E. quadrant overloaded	54,180
II-A	Signalization plus 2-lane exit ramp and double left turn for S. W. movement	5,991	Entrance ramp in S. W. quadrant overloaded	8,233	Signalization overloaded	59,980
II-B	Same plus 2-lane entrance ramp in S. W. quadrant	6,686	Signalization overloaded	8,233	Signalization overloaded	66,860
II-C	Same plus additional approach lane east and double left for E.-S. movement	9,000	Southbound lanes of I-4 overloaded	9,417	Northbound lanes of I-4 overloaded	90,000
II-D	Same plus widening I-4 to 6 lanes	9,855	Signalization overloaded	10,438	Signalization overloaded	98,550
III-A	Partial cloverleaf design; all ramps 1 lane	6,376		8,506	Exit ramp in S. E. quadrant overloaded	63,760
III-B	Same with extra lane on east approach of Fla-436	9,000	Southbound lanes of I-4 overloaded	8,506	Exit ramp in S. E. quadrant overloaded	85,060
III-C	Same with 2-lane exit ramp in S. E. quadrant	9,000	Southbound lanes of I-4 overloaded	9,416	Northbound lanes of I-4 overloaded	90,000
III-D	Same with I-4 widened to 6 lanes	9,197	Signalization overloaded	10,611	Eastbound lanes of Fla-436 overloaded	91,970
IV-A	Flyover design; all lanes 1 lane	9,000		8,506		85,060
IV-B	Same with 2-lane exit ramp in S. E. quadrant	9,000	Southbound lanes of I-4 overloaded	9,416	Northbound lanes of I-4 overloaded	90,000
IV-C	Same with I-4 widened to 6 lanes	9,456	Flyover ramp overloaded	12,804	Northbound lanes of I-4 overloaded	94,560
IV-D	Same with 2-lane flyover ramp	13,500	Southbound lanes of I-4 overloaded	12,804	Northbound lanes of I-4 overloaded	128,040
V-A	3-level diamond interchange	5,991	Signalization overloaded	5,977	Exit ramp in S. E. quadrant overloaded	59,770
V-B	Same with 2-lane exit ramp in S. E., 2-lane entrance ramp in S. W.; double left turn for E.-S. movement	8,913	Southbound lanes of I-4 overloaded	9,417	Northbound lanes of I-4 overloaded	89,130
V-C	Same with I-4 widened to 6 lanes	11,783	Entrance ramp in S. W. quadrant overloaded	10,810	Eastbound lanes of Fla-436 overloaded	108,100
VI	Relocation of 2-lane flyover ramp; I-4 to 6 lanes; 2-lane ramps in southern quadrants	11,982	Entrance ramp in S. W. quadrant overloaded	9,890	Signalization overloaded	98,900

ramp in the southwest quadrant. The capacity of this 1-lane entrance ramp and the left-turn capacity of the east-to-south movement are both potentially critical. In the afternoon peak hour, the heavy south-to-west movement makes the capacities of both the 1-lane exit ramp in the southeast quadrant and the left-turn movement off this ramp potentially critical. The improvements considered under candidate system II are relatively minor geometric changes to increase the capacities of these critical elements.

Candidate System II-A—The previous analysis indicated that the capacity of the exit ramp in the southeast quadrant limited the equivalent ADT of candidate system I. Consequently, candidate system II-A is similar to candidate system I but has a 2-lane exit ramp in the southeast quadrant. Because of the heavy left-turn volume using this ramp, a double left turn is also provided for those left turns. The shaded area shown in Figure 1 indicates the area changed from the previous candidate system; Table 1 gives the results of those changes. No changes occurred in the morning peak-hour capacity, for no attempt was made to increase it. However, the capacity of the afternoon peak period increased to 8,233 vph. The morning peak-hour capacity is now lower and determines the equivalent ADT, which is 59,910 vehicles per day.

Candidate System II-B—The entrance ramp in the southwest quadrant was the geometric element that limited the equivalent ADT in candidate system II-A. Consequently, in candidate system II-B this ramp is widened to a 2-lane ramp. All of the previous improvements are retained in this candidate system. The capacity of the morning peak period was increased to 6,686, and that of the afternoon peak period was unchanged. The improvement raised the equivalent ADT to 66,860 vehicles.

Candidate System II-C—In both peak periods under candidate system II-B, the capacity was restricted by the capacity of the signal system. Consequently, the improvement incorporated into candidate system II-C is to add an approach lane on Fla-436 from the east and to provide a double left turn for the east-to-south movement. This changes the critical lane volumes in the signalization constraint and increases the capacities of both the morning and afternoon peak periods to 9,000 vph and 9,417 vph respectively. The equivalent ADT is 90,000 vpd, and the capacity of I-4 south of the interchange limits the capacity in both the morning and afternoon peak periods.

Candidate System II-D—In candidate system II-D, I-4 is widened to 6 lanes. This will increase both the peak-period capacities. The morning peak-hour capacity is 9,855 vph, and the afternoon peak-hour capacity is 10,438 vph; the equivalent ADT is 98,550 vehicles per day. At this point in the analysis, the capacity of both peak periods is limited by the capacity of the signalization. All reasonable improvements in this area have been made, and candidate system II has been fully exhausted.

Candidate Systems III, IV, V, and VI—Each of these systems represents a new general configuration (Fig. 1 and Table 1).

Candidate System Capacities and Future Volume Projections

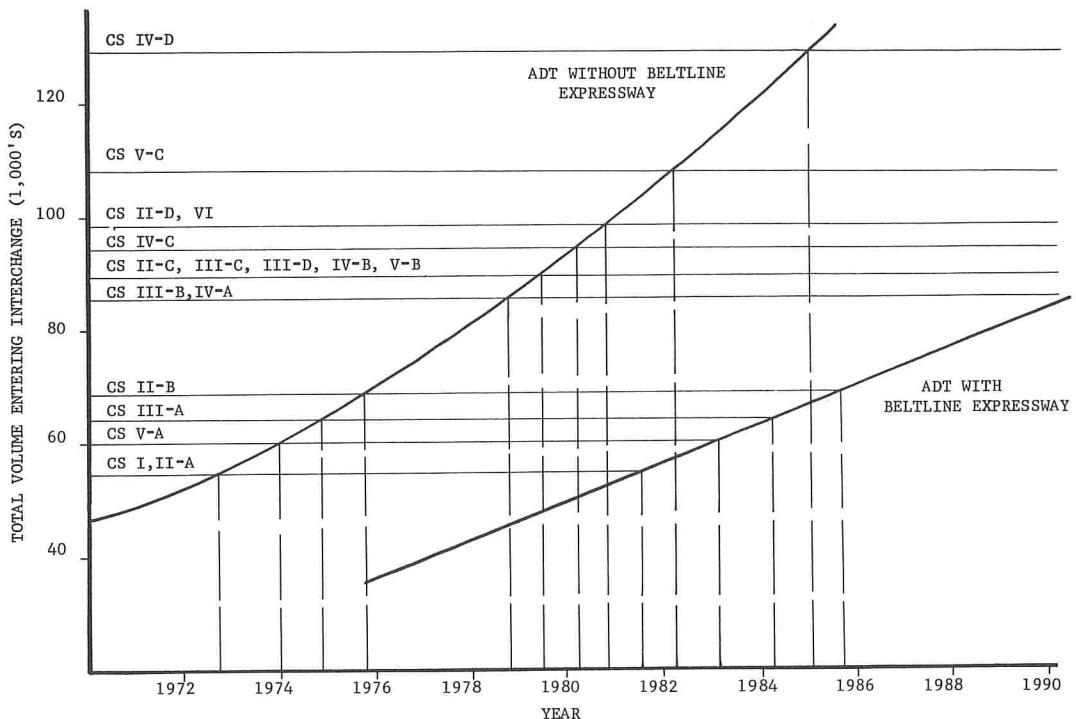
The relative effectiveness of each candidate system can be seen by plotting the projected future volumes and superimposing on this figure the defined 24-hour capacity for each system. This is shown in Figure 2. The 20-year forecast with and without the Beltline Expressway enables the designer to determine which candidate system will be capable of accommodating the future demands. The year in which each candidate system will fail to efficiently accommodate the expected demand is given in Table 2.

Cost Analysis

The cost of each of the alternative configurations was estimated by using prevailing unit cost figures that were provided by the Florida Department of Transportation. Table 2 gives a summary of the cost analysis and also indicates which of the configurations have the disadvantages of requiring substantial additional right-of-way or requiring major reconstruction of the interchange.

Cost-Effectiveness Analysis

A cost-effectiveness analysis is, in general, conducted to ensure that in the selection of a preferred system the greatest return is obtained for the capital invested. Cost-

Figure 2. Traffic demand projections and relation to equivalent ADT capacities of candidate systems.**Table 2.** Cost and capability of candidate systems.

Candidate System	Year Capacity Reached		Right-of-Way Required	Major Construction Required	Cost (\$)
	With Beltline Expressway	Without Beltline Expressway			
I	1982	1973	No	No	10,000
II-A	1982	1973	No	No	16,111
II-B	1985	1976	No	No	21,444
II-C	1990+	1979	No	Yes	95,460
II-D	1990+	1981	No	Yes	201,904
III-A	1984	1975	Yes	Yes	2,676,762
III-B	1990	1979	Yes	Yes	2,678,778
III-C	1990+	1979	Yes	Yes	2,684,828
III-D	1990+	1979	Yes	Yes	2,791,272
IV-A	1990	1979	Yes	Yes	2,136,368
IV-B	1990+	1979	Yes	Yes	2,142,418
IV-C	1990+	1980	Yes	Yes	2,248,862
IV-D	1990+	1985	Yes	Yes	2,276,382
V-A	1983	1974	No	Yes	1,382,316
V-B	1990+	1979	No	Yes	1,465,646
V-C	1990+	1982	No	Yes	1,572,090
VI	1990+	1980	No	Yes	1,359,364

effectiveness analyses are conducted in many different ways depending on the purpose of the original analysis. If it has already been decided to undertake the mission that requires one of the candidate systems, the cost-effectiveness analysis frequently involves the selection of the system that meets the system requirements at a minimum cost. This is frequently the case in military applications, but generally in civil engineering applications the specific value of the requirement is not quite so clear. Consequently, the analyses become more complicated and often involve trade-off analyses of the incremental return for an incremental investment.

The measure of effectiveness that is used to evaluate the performance of an interchange is the equivalent ADT or total (entering) capacity of the interchange, and the cost is the initial cost of the improvement (it would also be possible to use annual costs). Figure 3 shows the cost and effectiveness plot of the candidate systems. The equivalent entering ADT's projected for 1991 for the interchange are 78,600 if the Beltline Expressway is constructed and 157,500 if it is not. This will result in a level of service E during peak period. In order to obtain level of service C during peak period, these capacity values should be raised to 100,000 and 200,000 ADT.

The effect of the future of the Beltline Expressway is quite significant. Because the construction of the Beltline Expressway is uncertain, decisions on the improvements to the I-4 and Fla-436 interchange essentially represent gambles on the outcome of the Beltline Expressway. This is a classic case of decision-making under uncertainty, which is the subject of many books in the field of management science.

Figure 3 shows many interesting insights into the decisions involved. First, none of the candidate systems considered will provide sufficient capacity in the event that the Beltline Expressway is not constructed. Candidate system IV-D yields the highest capacity, and the capacity of the 6 lanes of I-4 is reached. To increase the capacity above that of candidate system IV-D would involve the reconstruction of I-4 to 8 lanes. Thus, every consideration should be given to an alternate facility in this rapidly growing area.

If we consider any particular candidate system, it would never be advisable to select a system that costs more and yields a lower capacity than system IV-D. With this in mind, we can greatly narrow the number of candidate systems to be considered. If we start the examination with the least expensive system, we can progressively consider larger expenditures if we consider only systems that produce an increase in capacity for the additional investment. Thus, the only candidate systems that must be considered are systems I, II-A, II-B, II-C, II-D, VI, V-C, and IV-D. Only systems II-D, V-C, VI, and IV-D produce capacities approximately equal to those required to yield level of service C during peak periods (assuming that the Beltline Expressway is built).

The next section presents the recommendations that involve an incremental approach to staging the improvements.

Recommendations

The recommendations that are presented represent a phased approach and are divided into 4 categories: immediate improvements, near-term improvements, medium-term improvements, and long-term improvements (necessary only if the Beltline Expressway is not constructed). Under immediate improvements, relatively inexpensive traffic control improvements are considered; under near-term improvements, minor reconstruction improvements are considered; under medium-term improvement, major construction items that can be made with no acquisition of right-of-way are considered; and under long-term improvements, an item that requires extensive acquisition of right-of-way is considered. The early improvements are also integral parts of later candidate systems.

Each of the improvements should represent one of the cost-effective candidate systems and should be compatible with 1 of the 4 candidate systems that will provide adequate future capacity. The specific improvements that are included in each candidate system can be identified to determine the compatibility of the various systems (Table 3). Candidate system I includes new signal controllers, and each of the other candidate systems also includes this feature; therefore, candidate system I is compatible with an incremental approach to any of the other candidate systems.

Candidate systems II-A and II-B would appear to contain the next increments of improvement—the reconstruction of the 2 ramps in the south quadrants as 2-lane ramps.

Figure 3. Cost-effectiveness of candidate systems.

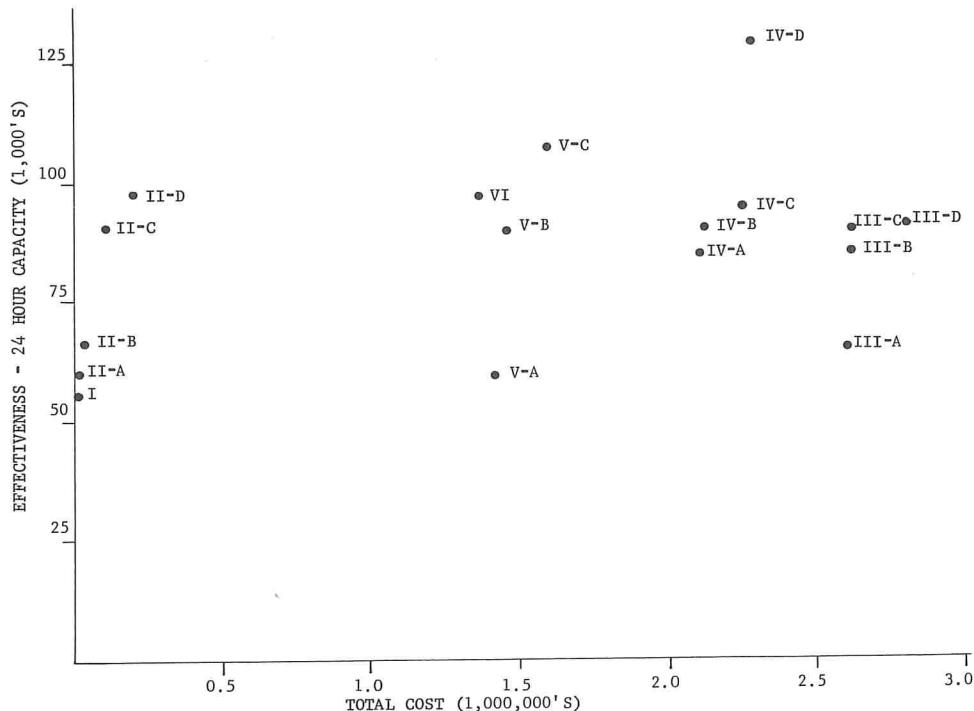


Table 3. Relation of cost-effective candidate systems and improvements.

Each of these improvements is an integral part of all other candidate systems. However, Figure 3 shows that candidate system II-C has a substantially higher capacity than candidate system II-B and has only a slightly higher cost. Because of this and because all the improvements in candidate system II-C are relatively minor reconstruction items, this system is selected as the one for the near-term improvements even though the double left-turn addition is not required in 2 of the more advanced systems. The incremental cost of the double left-turn improvement is low. The turn provides a sizable capacity addition and will permit deferring some of the more expensive improvements for a longer time in the future.

The next logical system is candidate system II-D, which involves the widening of I-4 to 6 lanes in addition to the previous improvements. At this point the equivalent ADT is 98,500 vehicles, which is adequate if the Beltline Expressway is built. Thus, if the expressway is built, no further improvements are necessary. If the expressway is not built, further improvements can be considered and candidate systems VI, V-C, and IV-D are the only ones that produce higher capacities. The capacity of candidate system VI is only slightly higher than that of candidate system II-D and would not warrant the large additional expenditures. Therefore, candidate system VI is not cost-effective. If the expressway is not built, its highest possible interchange capacity will be needed; and this will ultimately call for candidate system IV-D. Candidate system V-C is not suitable for phasing into candidate system IV-D, so the decision would be made to go directly to candidate system IV-D.

The recommendations can be summarized as follows:

Improvement Period	Candidate System	Years Required	
		With Expressway	Without Expressway
Immediate	I	Present-1982	Present-1973
Near term	II-C	1982-1990+	1973-1979
Medium term	II-D	Not needed	1979-1981
Long term	IV-D	Not needed	1981-1985

As a practical matter, candidate system IV-D would probably be built in 1979 if the expressway is not constructed and, thus, candidate system II-D would be bypassed.

CONCLUSIONS

The linear programming model of the interchange capacity and the cost-effectiveness analysis provides highway designers with a powerful tool to use in the selection of optimal interchange configurations and in the determination of a phasing program of future improvements. The designer can quickly consider many configurations, and the linear programming model, in fact, helps him determine the next logical configuration when the configuration under analysis has inadequate capacity.

ACKNOWLEDGMENT

The authors wish to acknowledge the support and active cooperation of the Florida Department of Transportation, sponsor of this research.

REFERENCES

1. Wattleworth, J. A., and Ingram, J. W. A Capacity Analysis Technique for Highway Junctions. Univ. of Florida, 1972.
2. Capelle, D. G., and Pinnell, C. Capacity Study of Signalized Diamond Interchanges. HRB Bull. 291, 1961, pp. 1-25.

DESIGN AND STRIPING FOR SAFE PASSING OPERATIONS

Graeme D. Weaver and John C. Glennon, Texas Transportation Institute,
Texas A&M University

ABRIDGMENT

•THE passing maneuver is one of the most hazardous operations on 2-lane highways. It is one of the few situations in which a driver may legally operate in the left lane of a 2-lane highway and, in so doing, may create a potential head-on collision. Yet, faster vehicles should be able to safely pass slower vehicles if efficient highway operations are to be maintained.

To provide the passing driver adequate sight distance and passing distance, we must assess the elements of the maneuver from a safety viewpoint and combine the critical elements in a compatible design. What is the critical condition in a passing maneuver? Is it a completed pass or an aborted pass? What distances are traveled during the perception-reaction time by the passing vehicle that occupies the left lane or by an opposing vehicle? At what point in the maneuver does the passing driver need the greatest sight distance? What "design speed" should be used? The answers to these questions are the inputs for formulating design and striping standards for safe passing sight distance.

The goals of this study were to examine passing behavior on rural 2-lane highways; to compare study parameters with the current passing sight distance design standards; and to develop, where appropriate, design and striping standards compatible with current operating conditions. Of primary concern were passing maneuvers on highways with operating speeds of 50 to 80 mph.

Current standards for designing passing sight distance and for striping rural 2-lane highways to restrict passing are based on different criteria. Passing sight distance is designed by using "A Policy on Geometric Design of Rural Highways," whereas no-passing zones are set by using the "Manual on Uniform Traffic Control Devices for Streets and Highways" (MUTCD). Unfortunately, the striping operation is done after the highway is constructed, when alignment changes are economically unfeasible.

The current standards for design and striping were critically evaluated with particular emphasis given to the inequities between design and operations. From this evaluation, and based on the operating characteristics of the passing maneuvers observed in the field studies, a new concept was developed that integrates design and striping to accommodate the safety and operational aspects of the passing maneuver.

SIGHT DISTANCE REQUIREMENTS

Current design standards are based on studies conducted from 1938 to 1941. The minimum passing sight distances for 2-lane highways were determined as the sum of 4 elements. From these studies of actual passing maneuvers on rural highways, distance values were established for the 4 elements of the maneuver: the perception-reaction distance, d_1 ; the left-lane distance, d_2 ; the clearance distance, d_3 ; and the distance traveled by an opposing vehicle, d_4 .

Once he has started a passing maneuver, the driver has only 2 options: complete the maneuver or abort the maneuver. If the passed vehicle maintains a constant speed, there is a point where the time to complete the maneuver is equal to the time to pull back.

This critical condition occurs about when the 2 vehicles are abreast. If at this point an opposing vehicle appears, the passing driver is forced to make a decision that affects the safety of the remaining portion of the maneuver.

The objective of passing sight distance design is to provide passing zones where maneuvers may be safely completed rather than aborted. Therefore, the critical completion distance is one of the elements to be included in the design. The distance required to complete the maneuver from the critical position is about $2/3d_2$. If the speed of the opposing vehicle and the passing vehicle are equal, the opposing vehicle also travels $2/3d_2$. If adequate clearance distance, d_3 , is included, the minimum sight distance required for safe operations is $4/3d_2 + d_3$.

The hazard associated with the passing maneuver arises when there is insufficient distance to complete the maneuver if an opposing vehicle is perceived at the critical position. The critical position can occur anywhere throughout the passing zone. To provide a safe "recovery zone" for the passing driver who faces the critical condition at the end of a passing zone, we must provide the minimum sight distance, $4/3d_2 + d_3$, throughout the passing zone. This philosophy approaches the long-zone passing concept because it provides a safe recovery area in a no-passing zone but does not encourage drivers to initiate an illegal passing maneuver.

DESIGN SPEED

A basic inequity between design and operations is that speeds lower than the design speed are used to compute the distance elements. For design speeds greater than 50 mph, the passing speed is assumed to be less than the design speed; this difference increases as the design speed increases.

New stopping sight distance standards are determined by assuming that the vehicle travels at the design speed. This approach is compatible with the "design" concept in engineering practice. Designing is more critical for the passing maneuver than for the stopping maneuver because the passing driver is maintaining a relatively high speed or accelerating. Yet, in the design of passing sight distance, the passing vehicle is assumed to be traveling at a speed less than the design speed.

Because the passing maneuver represents one of the most hazardous operations on a 2-lane highway, it is logical, from a critical design standpoint, that the sight distance elements be computed for a passing vehicle traveling at design speed. Also, so that all elements of the maneuver are placed on a common basis, the opposing vehicle should also be considered to be traveling at design speed.

PASSING ZONE LENGTHS

Passing sight distance design is determined on the basis of sight distance between 2 vehicles approaching each other at opposite sides of a crest vertical curve. A more common situation occurs when sight distance on one crest is limited by the next successive crest in rolling terrain. Often, the driver experiences a series of short passing zones through the sags and is immediately faced with a no-passing zone as he approaches each crest. No provision is made in the current design standards to prohibit this occurrence. These standards specify that certain sight distances be provided for particular design speeds but do not specify the length over which that sight distance must be made available. In other words, a section of highway could have the required sight distance at the crest of a vertical curve, and very shortly thereafter the available sight distance could decrease to less than the design requirement.

Currently, the length of passing zones or the minimum distance between successive no-passing zones is specified as 400 ft in the MUTCD. This distance is not sufficient for modern high-speed passing maneuvers. Limited studies of short passing sections on main rural highways have shown that most drivers do not complete a pass even within an 800-ft section. Actually, the drivers who passed in those short sections were often in the critical position beyond the passing zone where sight distance was less than minimum. A desirable minimum length of passing zone includes the perception-reaction distance, d_1 , and the left-lane distance, d_2 . If the maneuver is initiated at the beginning of the zone, this distance permits the driver to abort the maneuver if an opposing ve-

hicle is perceived before reaching the critical position. This length also permits the completion of a maneuver within the passing zone if the opposing vehicle is perceived past the critical position.

FIELD STUDIES

A movie camera mounted in an observation box on the bed of a pickup truck was used to photograph passing maneuvers at 3 study sites. Passing situations were created with an impeding vehicle traveling at a predetermined speed. The observation vehicle moved in behind a subject vehicle about 2 miles upstream from the study site. As the 2 vehicles approached, the impeding vehicle stationed on the shoulder near the beginning of the no-passing zone preceding the study site moved out and impeded the subject vehicle. Filming was initiated as the 3 vehicles reached the study site. Approximately 3,000 subjects were tested. Of this number, about 500 completed passing maneuvers were filmed. Impeding speeds were 50, 55, 60, and 65 mph.

Each study site was marked with stripes placed perpendicular to the centerline at 40-ft intervals. This reference system allowed the determination of the speed and distance elements of the passing maneuver by analyzing the film on a Vanguard motion analyzer. Cumulative percentiles of measured speed differentials were plotted for each impeding speed. The 15th percentile was selected as the critical condition. This critical differential was found to decrease as impeding speed increased, ranging from about an 11-mph differential at 50 mph to a 7-mph differential at 65 mph.

Eight best-fit relations were obtained by plotting passing speed against the distance elements d_1 and d_2 for each of the 4 impeding speeds. The relations between each of these distance elements and design speed were then obtained by a best-fit plot through the 4 points representing the distance element at the passing speed equal to the impeding speed plus the speed differential. Those relations were similar to those used in current passing sight distance standards.

IMPLEMENTATION

Table 1 gives the proposed passing sight distance and passing zone length standards for designing and striping passing zones. These values are based on the analysis of the field measurements using the proposed design concept. Examination of the proposed standards reveals several important factors to consider for passing sight distance design. For every design speed, the passing sight distance at the beginning of the zone exceeds the current AASHO standard. The available sight distance at the beginning of a zone is determined by establishing the end of the passing zone, and that is done by finding the point on the profile where sight distance is limited to $4/3d_2 + d_3$; then the beginning of the passing zone is located upstream from this point a distance equal to or greater than the minimum passing zone length of $d_1 + d_2$. The sight distance at the beginning of the zone must, therefore, be at least the sum of these 2 distances, or $d_1 + 2.33d_2 + d_3$.

The 70-mph design speed is used to illustrate another design consideration given in Table 1. If the spacing between successive crests is greater than 3,310 ft, adequate

Table 1. Proposed standard for design and striping passing zones.

Design Speed (mph)	Minimum Sight Distance Through-out Zone (ft)	Minimum Desirable Length of Passing Zone (ft)	Minimum Sight Distance at Beginning of Zone (ft)	Design Speed (mph)	Minimum Sight Distance Through-out Zone (ft)	Minimum Desirable Length of Passing Zone (ft)	Minimum Sight Distance at Beginning of Zone (ft)
50	1,135	885	2,020	70	1,825	1,485	3,310
60	1,480	1,185	2,665	75	2,000	1,785	3,635
65	1,655	1,335	2,990	80	2,170	1,935	3,955

sight distance and passing zone length are automatically provided in the sag. If, however, the distance is slightly less than 3,310 ft, and neither crest affords 1,825 ft of sight distance, an adequate passing zone does not exist. In this case, a passing zone can be provided by minor adjustments to the grade lines.

Historically, vertical profiles have been established by the economic considerations of earthwork. Although the balance of cut and fill is important in establishing profile, it is possible that a substantial improvement in traffic efficiency may be attained by minor adjustments in grade. Flattening grade lines in a sag, in effect, moves both crests outward.

From these considerations, proper passing sight distance in gently rolling terrain is clearly influenced by profile establishment. Computer programs are used widely to establish profile. Cost-effectiveness techniques can be incorporated to determine the benefits derived from grade adjustments for reasons other than earthwork balance.

Another consideration in design is the determination of optimum lengths of passing zones. Limited studies have indicated that utilization is very low for passing zones shorter than about 900 ft based on the current MUTCD standard of 1,200-ft sight distance. Obviously, there exists a passing zone length that many drivers will consider too short for a safe passing maneuver. Additional research is obviously warranted to provide the necessary data for cost-effectiveness evaluations.

INSTABILITY ANALYSIS OF A VEHICLE NEGOTIATING A CURVE WITH DOWNGRADE SUPERELEVATION

William Zuk, Consultant to the Virginia Highway Research Council

This study was initiated as a result of numerous skidding accidents occurring at locations where highway geometrics include a combination of downgrade, curve, and superelevation. Mathematical equations are developed for obtaining all the wheel forces (both normal and lateral) of a vehicle negotiating such a curve at the instant of incipient skidding for a variety of parameters. Factors that appear to be most important in regard to critical skidding velocities are the lateral coefficient of friction between the tire and the road surface and driver maneuvering. Factors that appear to have little influence are superelevation (if relatively small), crosswind velocity, and type of vehicle (excluding tractor-trailers).

•ON INTERSTATE 95, at the interchange with US-1, numerous skidding accidents have occurred in recent years. As the tested coefficient of friction between treaded tires and the road surface at this location is well above 0.4 (even when wet), other reasons for the many skids were investigated. At this site there is a 1-deg horizontal curve, a downhill grade of 2.6 percent, and a transverse superelevation of 0.0156 ft/ft. The combination of these geometric conditions has never been investigated for its effects on skidding; therefore, this mathematical study was initiated to obtain a quantitative means of predicting critical skid velocities for a variety of parameters. (As a result of observations at this site, other sites with downgrade superelevation have been inspected and also found to be associated with numerous skidding accidents. This finding suggests that skidding on a downgrade superelevation is not an isolated problem but perhaps a general one requiring special attention.)

The various parameters studied include highway grade, superelevation, vehicle weight, vehicle geometry, crosswind velocity, road surface condition (coefficient of friction), and driver correction maneuvers (nontracking of the intended roadway path).

ANALYSIS

Figures 1 and 2 show the free body diagrams of the vehicle projected in 2 vertical planes. Included are the centrifugal D'Alembert forces caused by the vehicle negotiating a downhill curve to the right. It is assumed that the center of mass will not shift its relative position appreciably because of the dynamic forces acting on the elastically sprung body mass.

Also, for simplicity, the lateral wind force L is assumed to act through the center of mass, which is justified on the basis that, at the high vehicle speeds at which skidding takes place, the wind force is but a very small fraction of the total lateral force.

Other notations are defined as follows:

W = gross weight of vehicle;

g = acceleration due to gravity (32.2 ft/sec^2);

R = radius of curvature of vehicle path (this value may or may not coincide with radius of road, for a driver maneuver within the width of the roadway could control the value of R used);

Figure 1. Forces and dimensions in vertical plane (rear view).

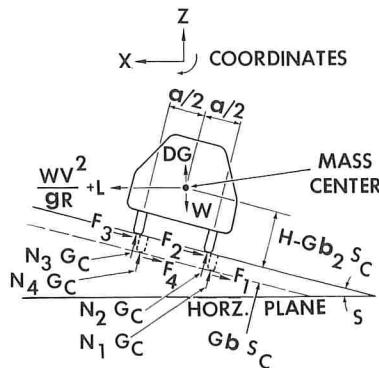


Figure 2. Forces and dimensions in vertical plane (side view).

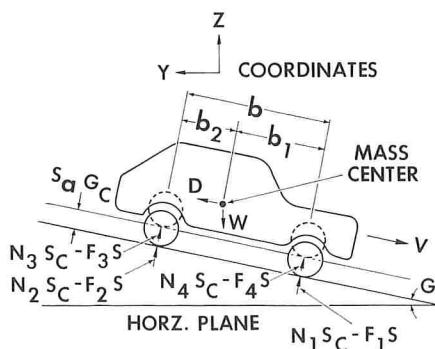


Table 1. Critical speeds for sample cases.

Case	Maneuver	R (ft)	Cross- wind (mph)	Condition Change	Critical V (mph)
1	Tracking curve		0		164.6
2	Tracking curve		40*		151.0
3	Tracking curve		40	Icy pavement, $m = 0.1$	74.9
4	Corrective maneuver	1,000	0		68.7
5	Corrective maneuver	1,000	40		63.0
6	Corrective maneuver	1,000	40	Superelevation of 0	61.2
7	Corrective maneuver	1,000	40	Superelevation of 0.0312 on 1	64.9
8	Corrective maneuver	1,000	40	$m = 0.4$	74.0
9	Corrective maneuver	1,000	40	$m = 0.5$	83.5
10	Corrective maneuver	1,000	40		61.4
11	Corrective maneuver	1,000	40		63.7

^aBlowing from inside of curve to outside.

L = lateral force due to wind on the vehicle (approximately equal to the product of 0.00254, lb, the projected side area of the vehicle, ft^2 , and the crosswind velocity, mph^2);
 D = aerodynamic drag of vehicle (assumed to act through center of mass);
 H = height of center of mass of vehicle above a level road surface;
 N_1, N_2, N_3, N_4 = normal forces on wheels;
 F_1, F_2, F_3, F_4 = lateral forces on wheels;
 G = grade of road, radian (small angles assumed);
 S = superelevation of road, radian (small angles assumed);
 G_c = cosine G ;
 S_c = cosine S ;
 V = velocity of vehicle;
 m = coefficient of side slip friction on right front wheel; and
 m_2, m_3, m_4 = coefficient of side slip friction on other wheels.

The following dynamic equations can be written (assuming steady-state dynamic conditions). The horizontal equation in the X-direction is

$$(WV^2/gR) + L - SG_c(N_1 + N_2 + N_3 + N_4) - S_c(F_1 + F_2 + F_3 + F_4) = 0 \quad (1)$$

The vertical equation in the Z-direction is

$$- W - S(F_1 + F_2 + F_3 + F_4) + S_c G_c (N_1 + N_2 + N_3 + N_4) + DG = 0 \quad (2)$$

The DG term is equal to WG^2 (from the constant V relation), which is very small and is neglected. The roll equation about the Y-axis is

$$- (F_1 + F_4)(H + Gb_1 G_c) - (F_2 + F_3)(H - Gb_2 G_c) + (aG_c/2)(N_3 + N_4 - N_1 - N_2) = 0 \quad (3)$$

The pitching equations are (for 0 acceleration)

$$N_2 = (b_1/b_2)N_1 \quad (4)$$

$$N_3 = (b_1/b_2)N_4 \quad (5)$$

The yawing equations are (for 0 acceleration)

$$(b_1/b_2)F_1 = F_2 \quad (6)$$

$$(b_1/b_2)F_4 = F_3 \quad (7)$$

To solve for the 8 unknown wheel forces requires an additional relation provided by the slip equation

$$F_1 = mN_1 \quad (8)$$

As determined by Saibel and Tsao (1) and based on the sprung vehicle system, the first wheel to skid on cornering is the one on the right front.

The 8 equations listed can be solved simultaneously to obtain the tire forces as follows:

$$F_4 = (JWU + EAU - PTW)/(PM - QU) \quad (9)$$

$$N_4 = (TW + F_4 M)/U \quad (10)$$

$$N_1 = (N_4 SBG_c + F_4 BS_c - A)/J \quad (11)$$

$$F_1 = mN_1 \quad (8)$$

$$F_2 = (mb_1/b_2)N_1 \quad (12)$$

$$F_3 = (b_1/b_2)F_4 \quad (7)$$

$$N_2 = (b_1/b_2)N_1 \quad (4)$$

$$N_3 = (b_1/b_2)N_4 \quad (5)$$

where

$$B = (b_1 + b_2)/b_2;$$

$$J = B(-mS_c - SS_c);$$

$$E = B(-mS_c + S_c G_c);$$

$$A = (WV^2/gR) + L; \quad \text{where } W = \text{weight of vehicle}$$

$$P = BG_c(ES + JS_c);$$

$$T = -2Hm - (2b_1Hm/b_2) - aBG_c;$$

$$U = BG_c[TS_c - a(SBm + BS_c G_c)];$$

$$Q = SB(BmS_c - SBG_c) - EBS_c; \text{ and}$$

$$M = TSB - E[2H + 2Gb_1S_c + (2b_1H/b_2) - 2b_1GS_c].$$

These equations, arranged as they are in sequences, can easily be solved on a digital computer for any set of conditions.

To ascertain the critical skid speed, one merely programs the computer to start with some lower limit velocity (such as 40 mph) and to calculate the value of

$$m_4 = F_4/N_4 \quad (13)$$

for successively higher values of V (at small increments) until the value of m_4 equals or just barely exceeds the assigned value m . m_4 is the coefficient of friction of the left front wheel, the second wheel to initiate skidding, as determined by Saibel and Tsao (1). When $m_4 = m$, the entire vehicle may be assumed to be unstable, for the 2 rolling rear wheels offer no resistance against yawing. This criterion thus establishes the critical skid velocity.

SAMPLE SOLUTIONS

Although the developed equations are complete in themselves, 11 sample solutions or cases are presented to provide some physical interpretation for various typical conditions. The geometric conditions existing at I-95 and US-1 will be taken as standard, namely, a downhill grade of 2.6 percent, a superelevation of 0.0156 on 1, and a 1-deg curve to the right ($R = 5,730$ ft). For the basis of comparison, m will be taken as 0.3, except where noted; g is 32.2 ft/sec². Table 1 gives the maneuver being effected by the 3 types of vehicles and the critical speeds in relation to crosswind and vehicle path radius.

Cases 1 through 9 are for a standard American-made passenger car with the following properties: $W = 4,000$ lb (with passengers); $b_1 = 4.5$ ft; $b_2 = 5.5$ ft; $H = 2.0$ ft; $a = 5.25$ ft; and a projected side area = 50 ft².

Case 10 is for a small foreign car with the following properties: $W = 2,200$ lb (with passengers); $b_1 = 4.5$ ft; $b_2 = 3.5$ ft; $H = 1.6$ ft; $a = 3.7$ ft; and a projected side area = 35 ft².

Case 11 is for a large bus with the following properties: $W = 34,400$ lb (with passengers); $b_1 = 14$ ft; $b_2 = 8$ ft; $H = 5$ ft; $a = 8$ ft; and a projected side area = 390 ft².

CONCLUSIONS

Although the 11 cases given in Table 1 are but samples, some conclusions can be drawn concerning those factors that appear to be significant and those that do not.

Among the most significant factors is driver control. In case 1 a vehicle correctly tracking the curve skids at 164.6 mph (far above normal highway driving speeds and perhaps even beyond the possible speed of the vehicle itself), whereas in case 4 a slight corrective movement or swerve in the direction of the curve decreases this skid speed to 68.7 mph (close to the posted speed of 65 mph).

Another important factor, which is under the direct control of highway engineers, is the coefficient of friction. A comparison of cases 5, 8, and 9 shows that by increasing this coefficient critical speeds can be increased from 63.0 mph for $m = 0.3$ to 74.0 mph for $m = 0.4$ to 83.5 mph for $m = 0.5$, the last being well beyond the range of the posted speed limit. (Case 5 is a fairly severe condition, but not an improbable one. Skidding in this case occurs below the posted limit of 65 mph.) An icy road, for which the coefficient is of the order of 0.1, is of course a serious condition as can be seen by comparing cases 2 and 3. Ice on this roadway approximately halves the critical speed.

Parameters that appear to have relatively little influence are crosswind, degree of superelevation, and type of vehicle. The effect of crosswind is shown by comparing cases 1 and 2 or cases 4 and 5. In each set of cases, a fairly brisk crosswind of 40 mph decreases the critical speed by only about 7 percent. The influence of superelevation is illustrated by cases 5, 6, and 7. At the existing superelevation, the critical velocity is 63.0 mph; at 0 superelevation, the critical speed is 61.2 mph; and at double the existing superelevation, the critical speed is increased to only 64.9 mph. (All of these superelevations are relatively small.) For the 3 entirely different vehicles, cases 5, 10, and 11 show that the critical speeds are all practically the same (63.0, 61.4, and 63.7 mph respectively). Because of limitations in the formulation of the given theory, these conclusions apply only to 4-wheel, solid-body vehicles and not to tractor-trailer vehicles that possess additional wheels and additional degrees of freedom because of the pin-coupling between the tractor and trailer.

As all theories should be corroborated with tests, it is recommended that these conclusions be further investigated by physical tests with either small-scale models or full-scale vehicles. Needed in particular are data on side slip friction and driver steering wheel reaction in rounding a curve. Additional theoretical studies could also be made of the dynamic behavior of coupled vehicles, such as tractor-trailer or car-camper units, as well as of various transient effects, such as braking, transition curves, road bumps, and wind gusts. Refinements to the "rigid body" theory presented could also be made by including the dynamic effects of the springing of the vehicle itself.

Nonetheless, pending such further studies, it is believed that the results of this current study are sufficiently valid to suggest that all high-speed, curved highways with downgrade superelevation possess high friction capabilities under all weather conditions. The specific amount needed can be determined for any given site condition by the solution of Eqs. 8 through 13 along with Eqs. 4, 5, and 7 as described in this paper.

ACKNOWLEDGMENTS

Credit for assistance in this study is given to the following staff members of the Virginia Highway Research Council: David Mahone called this problem to the attention of the author and provided certain data; William Carpenter prepared the computer program used; and Jack Dillard was responsible for providing the financial means with which to pursue this study. Appreciation is also extended to the staff personnel who typed and prepared the paper in its final form.

REFERENCES

1. Saibel, E., and Tsao, M. C. Further Investigations Into Vehicle Dynamics. Society of Automotive Engineers, SAE Paper 100173, 1969.
2. Glennon, J. C. State of the Art Related to Safety Criteria for Highway Curve Design. Texas Transportation Institute, Texas A&M Univ., Res. Rept. 134, Nov. 1969.
3. Kett, I. Horizontal Highway Curve Practices in the U.S. and Western Europe. Consulting Engineer, May 1971, pp. 112-114.

AUTOMOTIVE NOISE: ENVIRONMENTAL IMPACT AND CONTROL

B. Andrew Kugler and Grant S. Anderson, Bolt, Beranek and Newman, Inc.,
Canoga Park, California, and Cambridge, Massachusetts

The steady increase in ambient noise levels in cities and suburbs is quickly becoming an important factor in the pollution of the environment. Among all sources of urban noise, automotive noise is the most widespread and important factor contributing to this increase. This paper discusses the effects of noise on people, the characteristics of automotive noise, and a method available to highway designers and engineers by which the environmental impact of the proposed highway on the surrounding community can be predicted. The paper concludes with a discussion of noise control through highway design. A quantitative example of such a method is discussed.

•THE PROBLEMS of environmental conservation and improvement are receiving a steady increase in public attention. Society is deeply concerned with the deteriorating characteristics of our once clean, quiet, and pleasant environment. Maintenance of clean air and water, control of noise, conservation of land resources and disposal of solid wastes are only a few of the problems facing us in the 1970's.

The steady increase in the ambient noise levels in cities and suburbs is quickly becoming an important environmental factor. Among all sources of urban noise, motor vehicle noise is the most widespread and important. Except for localized areas near airports or large industrial centers, motor vehicle noise is the controlling factor in setting the background noise levels in the community.

In this paper, we first discuss briefly the units used to describe automotive noise and the current criteria used in assessing the impact of traffic noise on people. Second, we discuss the various automotive sources that combine to create traffic noise. We close with a discussion on control of automotive noise. An example of noise control through highway design is presented.

UNITS AND CRITERIA USED TO RATE AUTOMOTIVE NOISE

The 3 characteristics of environmental noise that are of particular interest in determining subjective response are intensity or level of the sound, frequency spectrum of the sound, and time-varying character of the sound.

Characteristics 1 and 2 are adequately handled in the case of traffic noise by measuring or calculating the sound in terms of the A-weighted sound level. The A-scale reading of a standard sound-level meter provides a single-number measure of the noise stimulus and weights the frequency spectrum of the noise in accordance with subjective sensitivity to sounds of different frequency. As a convenient indication, the symbol dB (A) or dBA is used to refer to the A-weighted sound level.

Characteristic 3 reflects the fact that automotive noise is rarely constant; it changes from second to second, from minute to minute, and from hour to hour, following the effectively random details of traffic patterns. These time variations in noise levels are most sensibly accounted for statistically. The statistical time distribution identifies each level and the percentage of time when that level, during the short term, is exceeded.

Thus, the 50 percent level (or median level) identified by the symbol L_{50} is the level that is exceeded for 50 percent of the time. The 10 percent level is the level that is exceeded 10 percent of the time and is identified by the symbol L_{10} .

As related below, these 2 measures of the statistical time distribution are currently used both to develop criteria for automotive noise and to describe traffic noise situations. Other statistical measures used to rate automotive noise have been devised in recent years and include the Noise Pollution Level (1, 2) and the Traffic Noise Index. However, for the purposes of this discussion, we will use the L_{50} and L_{10} descriptors.

Automotive noise causes annoyance and dissatisfaction and interferes with tasks such as sleep, speech, and learning. Unfortunately, there is no satisfactory objective measure of the subjective effects of noise. This is primarily due to the wide variation in individual differences in the thresholds of annoyance, the habituation to noise as related to differing past experience with it, the semantic content of specific sounds, and the significance of the noise source itself.

In terms of task interference with speech and sleep, quantitative evaluations of criteria, while difficult, are more easily obtained. For example, data are available concerning the effects of steady masking noise on the intelligibility of speech in different environments or speech conditions. Information is also available on what noise levels are considered desirable in different speech environments. For example, what continuous noise levels should not be exceeded if adequate telephone use is to be expected, or what levels of noise permit acceptable TV listening for most people? The effects of noise on sleep interference is more difficult to assess because of the different physiological states of sleep and because sleep interference can exist without a person's being consciously awakened. However, recent experiments in this area have provided some guidelines for the selection of appropriate criteria in the case of automotive noise.

In general, the following can be deduced from laboratory and social survey studies:

1. Interference with speech and TV listening is the predominant complaint against automotive noise;
2. Interference with sleep is also often cited as a complaint; and
3. Measurements of both the time-average noise levels and the magnitude and frequency of occurrence of peak noise levels are important in describing subjective response to traffic noise.

On the basis of the considerations given above, suggested design criteria for traffic noise have been derived and are given in Table 1 (3). Those criteria specify maximum noise levels that would be considered by the average individual as acceptable with respect to speech, sleep interference, and annoyance for various community situations. Two criterion levels are given for each situation, one in terms of L_{50} levels and one in terms of L_{10} levels. In addition to the tabulated criteria, a limit on the increase in noise levels above prevailing ambient conditions is incorporated. Figure 1 shows how these criteria are used to evaluate the community impact of a proposed highway based on a given estimate of the highway noise levels and a measure of the prevailing ambient noise conditions.

AUTOMOTIVE NOISE SOURCES

What are the various sources responsible for automotive noise? How is this noise radiated and transmitted to the observer? Figure 2 shows that the automotive vehicle noise model (4) is indeed a complex one, containing numerous noise sources. Consider, for example, the engine casing as a noise source. Figure 2 shows that not only does the engine casing radiate airborne noise directly to the outside but also the engine vibrations generate noise in the form of structure-borne sound. Similarly, airborne noise generated under the hood is transferred into structural vibrations of hood and body. These vibrations, in turn, are again a source of airborne noise.

Another important source of automotive noise is the tire-roadway interaction. Figure 2 shows that, in addition to airborne and structure-borne noises produced by this source, we have to consider the problem of induced low-frequency, soil-borne vibrations. Tire-roadway induced seismic vibrations can be transmitted to nearby buildings, where vibrating floors, walls, and ceilings in turn can radiate sound to the observer.

Figure 1. Impact evaluation when predicted noise levels exceed criteria.

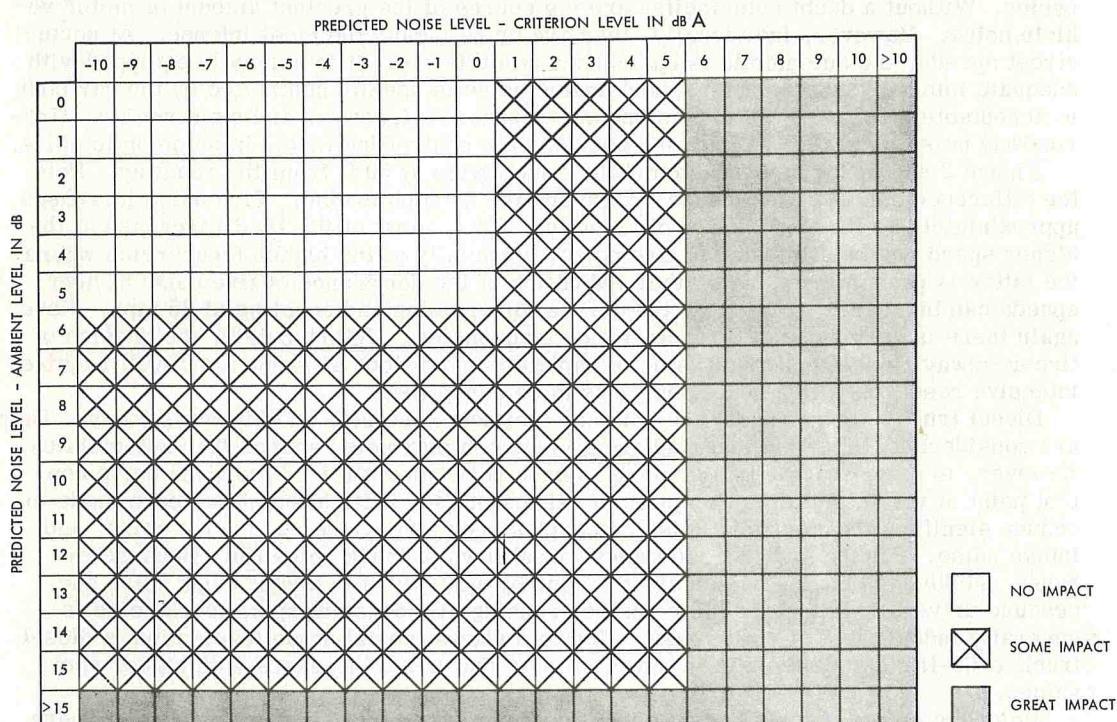


Table 1. Recommended design criteria.

Observer Category	Location	L ₅₀		L ₁₀	
		Day	Night	Day	Night
1	Residences ^a				
1	Inside	45	40	51	46
2	Outside	50	45	56	51
3	Schools ^a				
3	Inside	40	40	46	46
4	Outside	55	—	61	—
5	Churches				
5	Inside	35	35	41	41
6	Hospitals				
6	Inside	40	35	46	41
7	Convalescent homes				
7	Outside	50	45	56	51
8	Offices, stenographic				
8	Inside	50	50	56	56
8	Offices, private				
8	Inside	40	40	46	46
9	Theaters, movie				
9	Inside	40	40	46	46
9	Theaters, legitimate				
9	Inside	30	30	36	36
10	Hotels and motels				
10	Inside	50	45	56	51

^aEither inside or outside design criteria can be used depending on the utility being evaluated.

In a discussion of the noise of motor vehicles, it is advantageous to divide the vehicle population into 3 general categories, each having its peculiar characteristics: passenger vehicles and small trucks (here called automobiles), diesel trucks, and motorcycles. Without a doubt automobiles are the source of the greatest amount of motor vehicle noise. However, individually, they are by no means the most intense. At normal street speeds, the automobile is basically a quiet device. It is normally equipped with adequate intake exhaust mufflers, and engine noise is usually controlled by the car body to acceptable levels. However, as speeds increase on freeways and expressways, tire-roadway noise increases rapidly and becomes the controlling factor in automobile noise.

Figure 3 shows the typical automobile noise levels at 50 ft from the roadway. Note the difference between the 35-mph cruise and the 65-mph cruise. Tire noise increases approximately as the third power of velocity. Thus, most of the 10-dB increase at the higher speed can be attributed to tire noise, especially at the higher frequencies where the latter is predominant. A further indication of the dominance of tire noise at high speeds can be seen by comparing the curves for cruising and coasting at 65 mph. Here again there is very little difference at high frequencies. Unfortunately, the physics of tire-roadway noise mechanism is not completely understood at present and will require intensive research if this noise source is to be controlled.

Diesel trucks are generally the noisiest vehicles on streets and highways today. They are considerably larger in size and are a more complex noise source than automobiles. However, to date, tractor trailer designs have not been optimized from the noise control point of view. With the exception of exhaust noise, little attempt has been made to reduce significantly important noise categories such as gear noise, engine noise, and intake noise. Figure 4 shows one typical breakdown of truck noise (5). Because exhaust, intake, and engine noises are the controlling factors in diesel truck noise and because drivers tend to keep their engines running at a constant rpm, truck noise is generally independent of road speed. Figure 5 shows typical noise levels from a diesel truck at 50-ft distance from the roadway. Note that noise increases with higher rpm values.

Motorcycles are quickly becoming an important part of motor vehicle noise. There is an increasing dependence on this mode of transportation, especially among the young. Moreover, the minicycle or trail-bike is becoming an important source of high-intensity noise. In a recent survey of 48 different motorcycles (6) covering most of the popular makes sold in this country, noise measurements were performed under a number of test conditions. Surveyors used the proposed SAE test procedure in which noise measurements are made under a wide open throttle condition with the engine achieving its maximum rated rpm. It is interesting to note the general trend, shown in Figure 6, toward higher noise levels with increasing size. However, 3 of the largest motorcycles tested had noise levels well below the general trend. These results, together with our observations of motorcycle constructions, suggest that exhaust noise is often the controlling component and that exhaust mufflers supplied today on stock motorcycles exhibit a wide range of acoustical performance. This suggestion is reinforced by the large variation in noise levels measured (on the order of 10 dB) between the noisiest and quietest motorcycle in the 250-cc and 100-cc classes. Further proof of improper muffling can be found by comparing the levels of the 100-cc class with the 1,000-cc and higher class. The difference in muffling effectiveness is the more apparent when we consider that the larger machines have 10 times the displacement of the smaller.

The results of combining the various categories described above under different mix and speed conditions and in large numbers constitute what we generally call "traffic noise" or "automotive noise." Instead of being concerned with a single-event noise signal produced by a passing vehicle, we must consider the random noise produced by the superimposition of sound from many such vehicles.

In the simple drive-by case, it was appropriate to describe the event by its peak sound pressure level and spectrum shape. For flowing traffic, the temporal description of the random event must be considered by describing the signal statistically. Combining elements of traffic flow theory and knowledge of the noise characteristics of individual vehicles permits the generation of traffic noise models (1) that can be applied to the urban design problems. Using parameters such as traffic volume, average speed,

Figure 2. Automotive vehicle noise model.

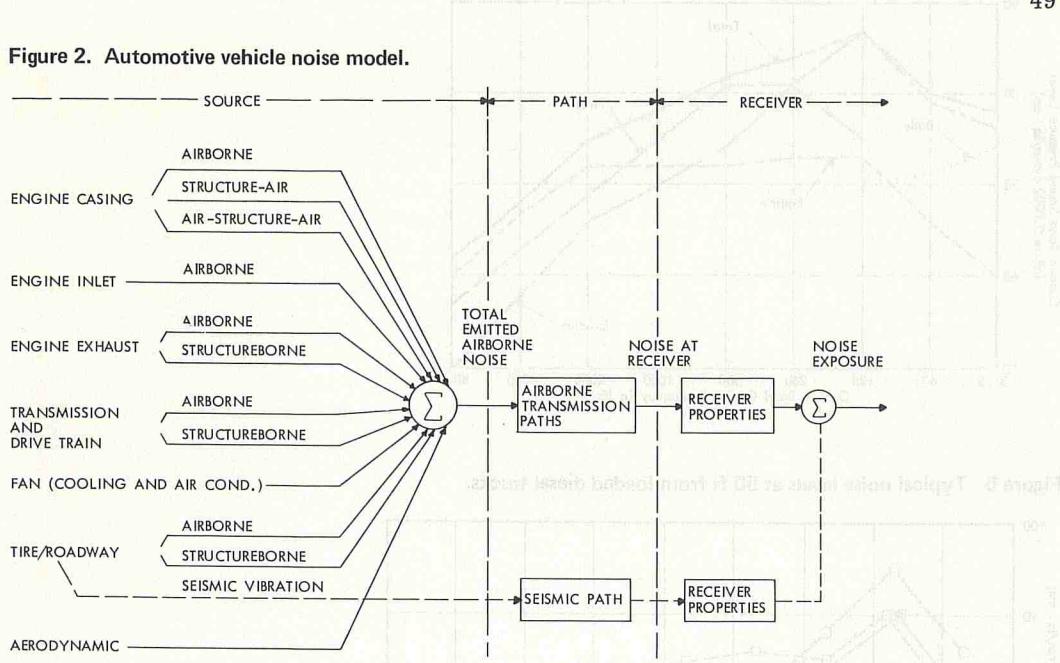


Figure 3. Typical noise levels at 50 ft from automobiles.

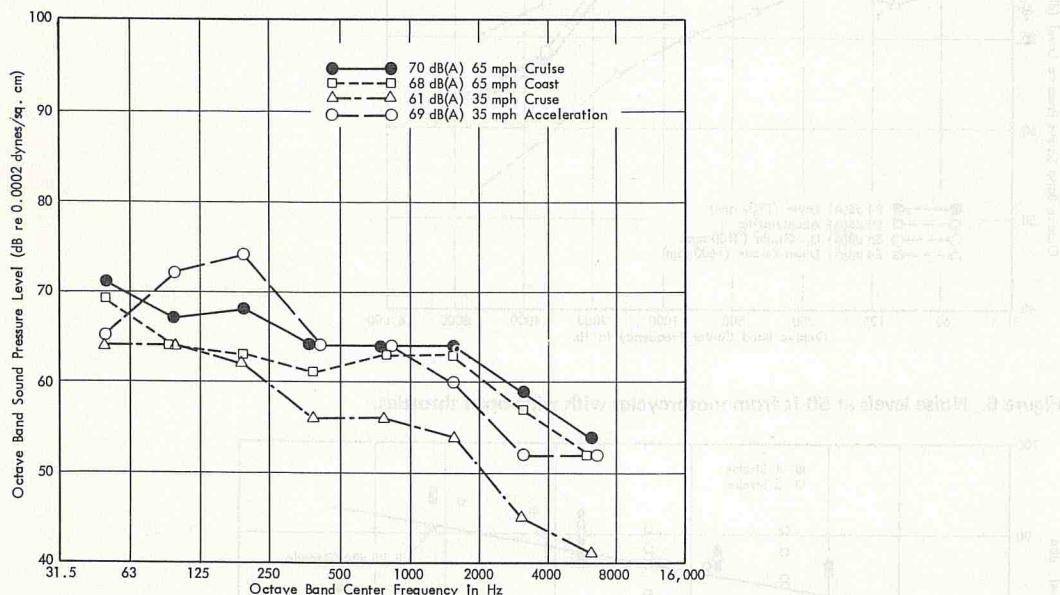


Figure 4. Typical noise sources in trucks.

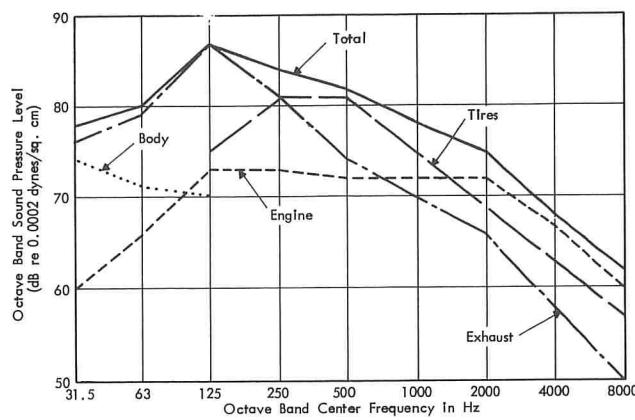


Figure 5. Typical noise levels at 50 ft from loaded diesel trucks.

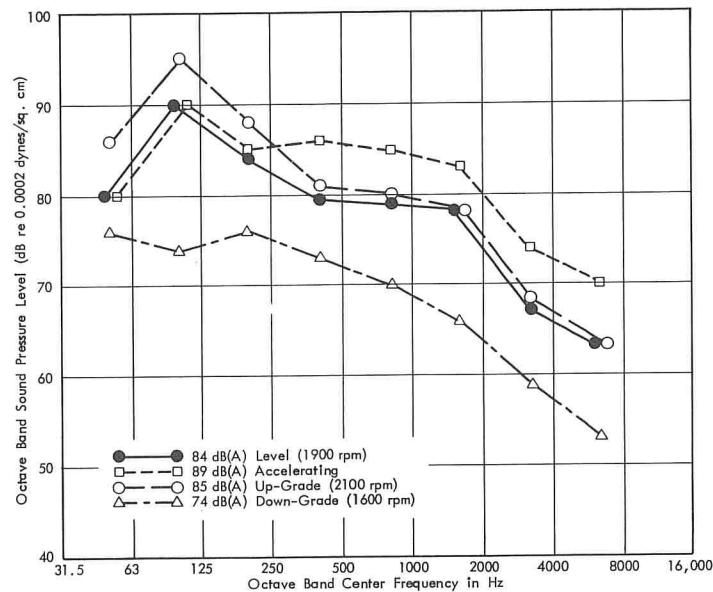
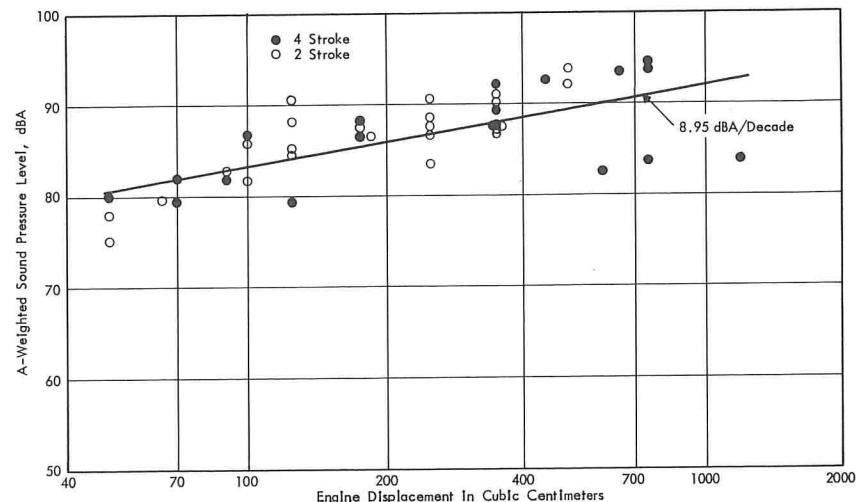


Figure 6. Noise levels at 50 ft from motorcycles with wide open throttles.



and automobile-truck ratio, we can predict the mean and variance of expected noise levels and their spectral distribution for many practical traffic situations of interest.

From these model studies and from actual traffic noise measurements near highways, estimates have been developed for traffic noise under different roadway conditions. Such estimates presented in terms of L_{50} sound pressure levels are shown in Figures 7 and 8 (3). As mentioned before, the L_{50} level represents the sound pressure level that is exceeded 50 percent of the time as measured in units of dBA. All that is needed to predict expected noise levels at 100 ft from the roadway is a knowledge of vehicle volume and average speed. Of course, these curves are based on a very simple traffic geometry. The roadway is represented by a straight, level, and infinitely long single lane. In order to predict the effect of different geometries on the expected noise levels and the changes at other distances from the roadway, adjustments to the basic model can be made, as will be shown later.

Figure 8 shows that truck noise seems to decrease with increased speed—an apparent contradiction of the earlier discussion. Actually, because we are concerned no longer with a single vehicle but rather with a statistical average description of a population of trucks, a constant vehicle flow represents a larger individual truck separation leading to a lower predicted noise level when average speed is increased.

NOISE CONTROL THROUGH HIGHWAY DESIGN

In the earlier discussion, we were concerned with the noise description and characteristics of motor vehicles and the manner in which those individual sources combine to create traffic noise. We now address the question of noise control. In general, and as shown by Figure 2, noise control can be achieved at 3 general points: at the source, at the receiver, and along the transmission path.

The first point involves control of noise produced by individual vehicles. Obviously, reducing the amount of noise emission at the source seems a logical and desirable approach. Research in this area is currently under way. Legislation limiting the noise levels of automotive sources has been adopted in some states. California and New York, for example, have quantitative noise limits for motor vehicles. Other states are sure to follow in the near future. However, much research is needed before further noise control at the source is significant. This is particularly true of the various sources of truck noise and of noise generated by all vehicles at the tire-roadway interface.

Noise control at the receiver can be achieved by careful planning and zoning of all new land uses close to major highways and arteries. Furthermore, through strong building codes, new buildings can be designed to achieve a satisfactory interior noise environment. At best, however, this method represents a solution to the noise problem only in particular cases. All people cannot retreat into insulated "boxes."

Noise control along the transmission path means reducing the noise levels between the source and the receiver. Here, there is much that can be done, especially in the case of a new highway. Accurate methods are available (3) for predicting the noise levels from a proposed highway route. Community response to various automotive noise levels can also be predicted by using criteria such as those given in Table 1. Therefore, noise trade-off studies can be included readily in the route selection and adjacent land planning processes. This in itself represents a good first step in highway noise control design.

However, in many cases new highways must pass through already densely populated areas. Here noise control can be achieved by detailed design of roadway configurations and alignments in order to minimize noise impact on adjacent land parcels and structures as was recently done for a roadway in Baltimore and for the Century Freeway in Los Angeles. By manipulating exact highway designs in local areas, the highway engineer or designer can vary the subjective loudness of future traffic noise by as much as a factor of 2 to 4. This is a major noise control accomplishment. To illustrate this possibility, let us consider the highway configuration shown in Figure 9. Here we are concerned with the impact of the proposed highway on the classroom noise environment of an elementary school. The school buildings are located approximately 400 ft from the closest point to the highway whose cross-sectional configuration is also shown.

What are the noise levels from the proposed configuration? At the time of interest, the average speed of traffic flow is estimated at 55 mph and the automobile-to-truck

Figure 7. L_{50} for automobiles as function of volume flow and average speed.

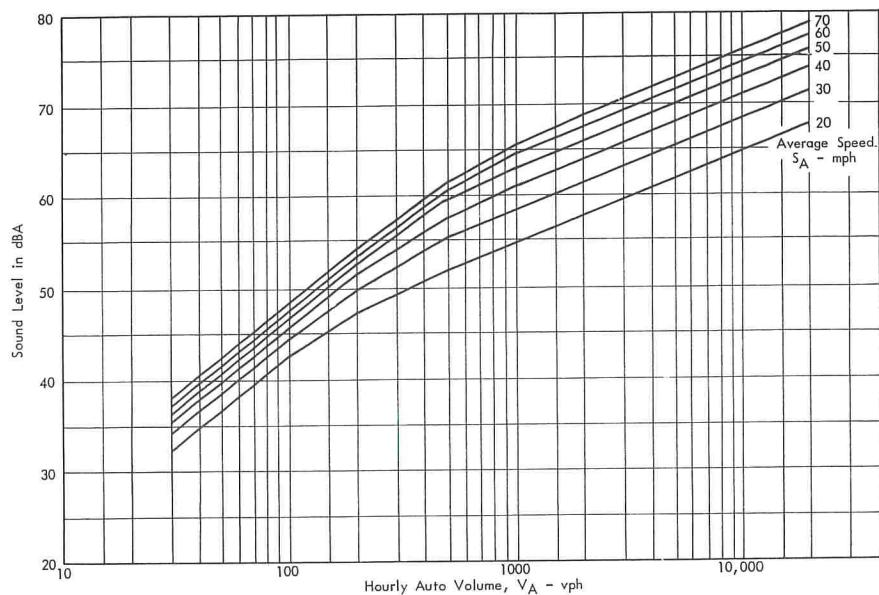


Figure 8. L_{50} for trucks as function of volume flow and average speed.

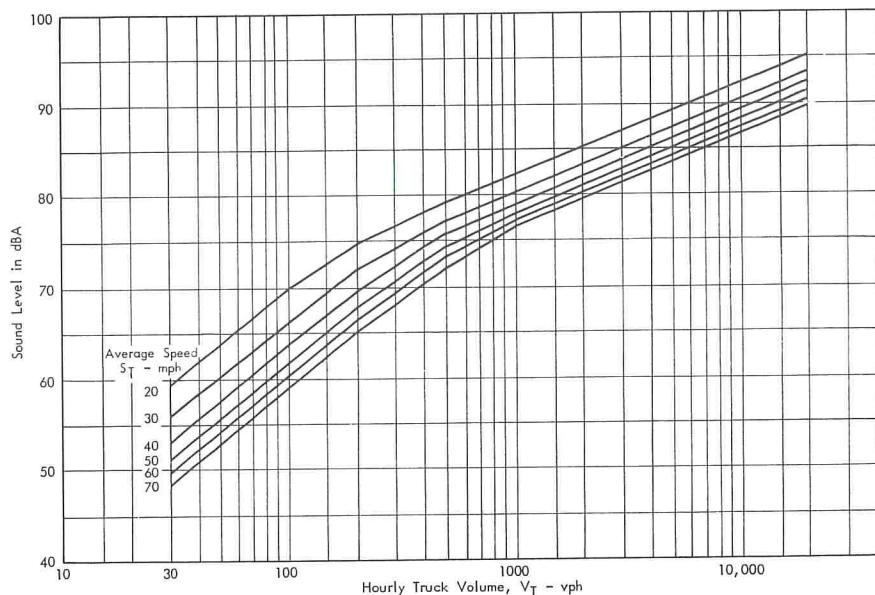


Figure 9. Example of proposed highway route.

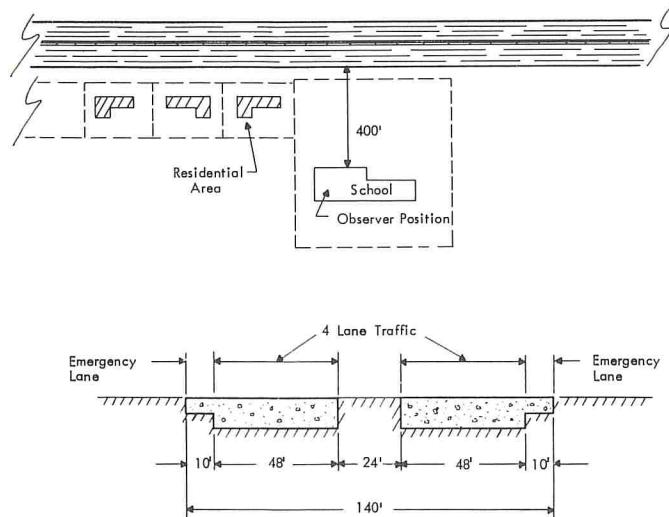
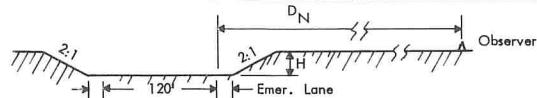


Table 2. Distance adjustment to account for observer-near lane distance and 120-ft roadway width.

Distance From Observer to Near Lane (ft)	Distance Adjustment (dB)	Distance From Observer to Near Lane (ft)	Distance Adjustment (dB)
50	0	600	-12
100	-2	700	-13
150	-5	800	-13
200	-7	900	-14
250	-8	1,000	-15
300	-9	1,200	-16
350	-10	1,400	-17
400	-10	1,600	-18
450	-11	1,800	-19
500	-11	2,000	-20

Table 3. Depressed roadway adjustment for short method only.

Depth of Depressed Roadway, H (ft)	Adjustment in dB at Distance from Observer to Near Lane, D _N						
	100 Ft	200 Ft	300 Ft	400 Ft	600 Ft	800 Ft	1,600 Ft
0	0	0	0	0	0	0	0
5	-6.0	-5.5	-5.0	-5.0	-5.0	-5.0	-5.0
10	-10.5	-10.5	-10.5	-10.5	-10.5	-10.5	-10.5
15	-13.0	-13.5	-13.5	-13.5	-13.5	-13.5	-13.5
20	-12.0	-14.0	-14.0	-14.0	-15.0	-15.0	-15.0
25	-11.0	-14.0	-15.0	-15.0	-15.0	-15.0	-15.0
30	-10.0	-14.5	-15.0	-15.0	-15.0	-15.0	-15.0
40	-9.0	-14.5	-15.0	-15.0	-15.0	-15.0	-15.0
50	—	-14.5	-15.0	-15.0	-15.0	-15.0	-15.0



ratio is 5 percent. In addition, it is estimated that the total vehicle volume is 7,000 vehicles per hour during the time of concern. As previously discussed and shown in Figures 7 and 8, we can readily obtain the expected L_{50} levels at 100 ft from the roadway. Figure 7 shows that a vehicle volume of 6,650 automobiles (95 percent of the total) traveling 55 mph will yield an L_{50} of 72 dBA at 100 ft. Similarly, 350 trucks per hour (5 percent of total) traveling at 55 mph will produce a noise level of 69 dBA. Because the school is located 400 ft from the highway, a distance adjustment is necessary. Table 2 shows that at 400 ft the levels given above will decrease by 10 dBA or to 62 and 59 dBA for automobiles and trucks respectively.

Data given in Table 2 indicate a 2 dBA adjustment at 100 ft. This is due to the inclusion of the roadway width into the calculations because the basic curves shown in Figures 7 and 8 assume a single-lane configuration. The data also show the influence of distance on the expected noise levels. Locating the highway twice as far (800 ft) from the school building will gain only 3 dBA more noise attenuation. Clearly then, distance alone is not a very effective means of noise control.

Data given in Table 1 show that the recommended design criterion for an inside environment in the school is 40 dBA. By adding logarithmically the L_{50} expected noise levels for automobiles and trucks, we obtain a total L_{50} level at the school building of 64 dBA. A good estimate of outside to inside noise reduction for a non-air-conditioned school is approximately 12 dBA. Thus, the L_{50} noise level due to the new highway inside the classroom is predicted to be 52 dBA. Clearly, this is not a satisfactory environment because our criteria recommended a maximum of 40 dBA. Thus, we can safely predict that the highway will have a negative impact on school environment. How can this situation be controlled and reduced to acceptable levels?

As mentioned above, distance is one means of control available. However, to achieve an additional 12-dBA noise reduction by this method would mean relocating the highway 2,400 ft farther away from the school. This in most cases is not an acceptable alternative. Other means of noise control involves depressing or elevating the highway from its original on-grade configuration. Table 3 gives the noise reduction that can be achieved by depressing the highway. Once again the figures apply only to the highway width and geometry indicated in our example. By depressing the roadway 15 ft below the on-grade condition, we find that an additional 13.5-dBA noise reduction is possible in the case of automobiles.

In general, the noise reduction for trucks is 5 dBA below that for automobiles. This is due to the difference in apparent source height. As mentioned before, the noise source in automobiles is close to the pavement while the source in trucks is elevated (exhaust stack). Thus in our case, a 15-ft depressed highway will lower the predicted noise levels to 41 dBA. A 25-ft depressed highway will further reduce the predicted noise levels to 39 dBA and satisfy our requirement. Based on the infinite length roadway model, this means depressing the roadway by 25 ft throughout its entire length or, practically speaking, for approximately 1 mile on each side of the observer.

Other approaches to noise control involve the use of earth berms or walls between the highway and observer or of partial shielding provided by other structures or of vegetation (not very effective). The choice of the most effective method of noise control for a particular situation depends on many factors. In our case we were concerned with the noise environment in the classroom only. This was done to simplify the example. However, in the general case, the analysis would necessarily also include the school's outside environment in the playground area as well as consideration of the nearby residential community. Only after carefully evaluating all factors involved, including the economics of possible solutions, can the proper method of noise control be chosen.

CONCLUSION

As illustrated by the example, control of highway noise through highway design is possible by proper planning. The inclusion of the noise considerations in all future highway designs can result, therefore, in a substantial control of this form of pollution. With a new awareness by the general public of the problems of noise pollution, the auto-

mobile manufacturer, the highway designer and engineer, and the city builder must all together meet the challenge and become seriously involved in the problem of automotive noise and its control.

REFERENCES

1. Robinson, D. W. The Concept of Noise Pollution Level. National Physical Laboratory, Aerodynamics Division, NPL Aero Rept. Ac38, March 1969.
2. Robinson, D. W. An Outline Guide to Criteria for the Limitation of Urban Noise. National Physical Laboratory, Aerodynamics Division, NPL Aero Rept. Ac39, March 1969.
3. Gordon, C. G., Galloway, W. J., Kugler, B. A., and Nelson, D. L. Highway Noise—A Design Guide for Highway Engineers. NCHRP Rept. 117, 1971.
4. Franken, P. A., and Bender, E. K. Environmental Effects of Transportation Noise. Trans., ASME, Paper 70-SF, 1970.
5. Anderson, G. S. Traffic Noise Computation. Bolt, Beranek and Newman, Inc., Lecture Series, Cape Cod, Sept. 14-10, 1970.
6. Noise Measurements of Motorcycles and Trucks. Bolt, Beranek and Newman, Inc., BBN Draft Rept. 2079, Feb. 1970.
7. Galloway, W. J. Clark, W. E., and Kerrick, J. S. Highway Noise: Simulation and Mixed Reactions. NCHRP Rept. 78, 1969.

SPONSORSHIP OF THIS RECORD

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
John L. Beaton, California Division of Highways, chairman

GENERAL DESIGN SECTION

F. A. Thorstenson, Minnesota Department of Highways, chairman

Committee on Geometric Highway Design

W. A. Wilson, Jr., North Carolina State Highway Commission, chairman
Glenn Anschutz, John E. Baerwald, Frederick G. Chatfield, P. B. Coldiron, John C. Glennon, C. William Gray, Peter J. Hunt, Jack E. Leisch, R. L. Lewis, D. W. Loutzenheiser, John Robert Moore, B. H. Rottinghaus, M. D. Shelby, Bob L. Smith, Kenneth A. Stonex, William P. Walker, J. Walter

PAVEMENT DESIGN SECTION

John E. Burke, Illinois Department of Transportation, chairman

Committee on Surface Properties-Vehicle Interaction

W. E. Meyer, Pennsylvania State University, chairman
Malcolm D. Armstrong, Glenn G. Balmer, F. Cecil Brenner, Arthur D. Brickman, William F. R. Briscoe, William C. Burnett, A. Y. Casanova, III, Blaine R. Englund, William Gartner, Jr., Ralph C. G. Haas, Douglas I. Hanson, Robert N. Janeway, David C. Mahone, B. F. McCullough, Robert B. McGough, Paul Milliman, Alexander B. Moore, Desmond F. Moore, E. W. Myers, F. William Petring, Bayard E. Quinn, John J. Quinn, Frederick A. Renninger, Rolands L. Ritzenbergs, Hollis B. Rushing, Richard K. Shaffer, Elson B. Spangler, W. E. Teske, M. Lee Webster, Ross G. Wilcox, Dillard D. Woodson

Lawrence F. Spaine, Highway Research Board staff

The sponsoring committee is identified by a footnote on the first page of each report.

THE National Academy of Sciences is a private, honorary organization of more than 800 scientists and engineers elected on the basis of outstanding contributions to knowledge. Established by a congressional act of incorporation signed by Abraham Lincoln on March 3, 1863, and supported by private and public funds, the Academy works to further science and its use for the general welfare by bringing together the most qualified individuals to deal with scientific and technological problems of broad significance.

Under the terms of its congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the federal government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the government.

The **National Academy of Engineering** was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its act of incorporation, adopted articles of organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the federal government, upon request, on any subject of science or technology.

The **National Research Council** was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to provide a broader participation by American scientists and engineers in the work of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial, and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities. Supported by private and public contributions, grants, and contracts and by voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

The **Division of Engineering** is one of the eight major divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

The **Highway Research Board** is an agency of the Division of Engineering. The Board was established November 11, 1920, under the auspices of the National Research Council as a cooperative organization of the highway technologists of America. The purpose of the Board is to advance knowledge of the nature and performance of transportation systems through the stimulation of research and dissemination of information derived therefrom. It is supported in this effort by the state highway departments, the U.S. Department of Transportation, and many other organizations interested in the development of transportation.

HIGHWAY RESEARCH BOARD
NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL
2101 Constitution Avenue Washington, D. C. 20418

ADDRESS CORRECTION REQUESTED

NON-PROFIT ORG.
U.S. POSTAGE
PAID
WASHINGTON, D.C.
PERMIT NO. 42970

